JOURNAL
OF
AMERICAN CONCRETE INSTITUTE

VOL. III MAY, 1915 No. 5

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PURPOSE OF THE INSTITUTE

It is the purpose of the Institute, by means of conventions and publications, to increase and disseminate knowledge in regard to the use of cement, concrete and allied products; to conduct research into their properties and uses; and to formulate recommended practice and standard specifications.

Its objects are purely educational.

JOURNAL
OF
AMERICAN CONCRETE INSTITUTE

FOUNDED 1905, AND INCORPORATED 1906, AS
NATIONAL ASSOCIATION OF CEMENT USERS
CHARTER AMENDED JULY 2, 1913

The Institute is not responsible, as a body, for statements and opinions advanced in its publications

VOL. III MAY, 1915 NO. 5

SELECTION OF MEMBERS OF THE COMMITTEE ON NOMINATION OF OFFICERS.

It was formerly the practice of this Institute for the Board of Direction to appoint the Committee on Nomination of Officers. This practice was objected to by some of the members of the Institute as not sufficiently democratic, and accordingly the By-Laws were amended so as to provide for the selection of this Committee by the members of the Institute.

The By-Laws provide that the Board shall canvass the opinions of the members on or before September 1st of each year. In accordance with this provision, there is enclosed a blank ballot, and it is expected that each member will send to the Board his suggestions for the members of this very important Committee.

The efficiency of the work of the Institute is dependent upon the interest and activity of all its members. There is no more important duty that each member should perform than that of voting for the members of the Committee which will nominate the candidates for the offices to be filled for the ensuing year.

RESULT OF LETTER BALLOT.

The recent letter ballot on the amendment to the By-Laws and on Standards, revision of Standards, and Recommended Practice, has resulted in their adoption.

Attention of the members is directed to the fact that beginning July 1st, the annual dues will be $10.00.

New Members.

The following new members have been elected:

GARDINER, WILLIAM H., Chief of Farm Bureau, Association American Portland Cement Manufacturers, Bellevue Court Bldg., Philadelphia, Pa.

HAWES, SUSAN M. (MISS), 287 South St., Morristown, N. J.

HISLOP, WILLIAM A., Engineer Pittsburgh Branch, Trussed Concrete Steel Co., 2124 Farmers Bank Bldg., Pittsburgh, Pa.

MCINTYRE, WILLIAM A., Chief Road Engineer, Association American Portland Cement Manufacturers, Bellevue Court Bldg., Philadelphia, Pa.

MECHANICS-MERCANTILE LIBRARY, F. B. GRAVES, Librarian, 57 Post St., San Francisco, Cal.

TONER, ARTHUR C., Road Engineer, Association American Portland Cement Manufacturers, Bellevue Court Bldg., Philadelphia, Pa.

Reduced Prices for Back Volumes.

Members of the Institute can obtain copies of back publications at the following special prices:

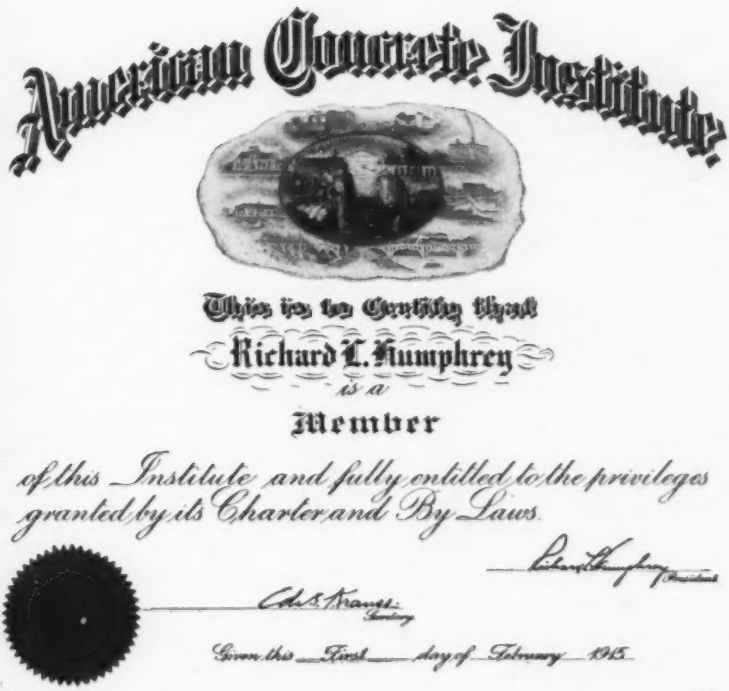
Vol. I, 50 cents; Vols. II to VI, $1.50 each; all in paper binding. For cloth or half-morocco binding there is an additional charge per volume of 50 cents or $1.00 respectively. Vol. VII is available only in cloth binding, $2.00, or half-moroco, $2.50.

Membership Certificate.

A Certificate of Membership in this Institute will be furnished to any member in good standing, i. e., whose dues are paid up to date. These Certificates are printed from an engraved steel plate. The name of the member is engrossed and the Certificate will be signed by the President and Secretary. The Certificates are about 10 x 14 ins., and have sufficient margin to be framed 14 x 17 ins. They are engraved on parchment or parchment paper, both suitable for rolling or framing.

The charge for certificates is as follows:

Parchment	$2.50
Parchment Paper	1.50


American Concrete Institute
This is to Certify that
Richard L. Humphrey
is a
Member
of this Institute and fully entitled to the privileges granted by its Charter and By Laws.
Given this First day of February 1915.

Standards Adopted by the Institute.

1. Standard Specifications for Cement with Appendix on Standard Methods of Testing and Chemical Analysis, 23 pp.
2. Standard Specifications for Portland Cement Sidewalks, 6 pp.
4. Standard Building Regulations for the Use of Reinforced Concrete, 13 pp.
5. Standard Specifications for One Course Concrete Highway, 10 pp.
6. Standard Specifications for Portland Cement Curb and Gutter, 7 pp.
7. Standard Recommended Practice for the Use of Reinforced Concrete, 21 pp.
8. Standard Specifications for Scrubbed Concrete Surfaces, 3 pp.
9. Standard Recommended Practice for Concrete Drain Tile, 3 pp.
10. Standard Recommended Practice for Concrete Architectural Stone, Building Block and Brick, 4 pp.
11. Standard Specifications for Concrete Architectural Stone, Building Block and Brick, 3 pp.
12. Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick, 3 pp.
13. Standard Specifications for Plain Concrete Floors, 6 pp.
14. Standard Specifications for Reinforced Concrete Floors, 5 pp.
15. Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Block, 9 pp.
16. Standard Methods for the Measurement of Concrete Work, 7 pp.
17. Standard Specifications for One Course Concrete Street Pavement, 10 pp.
18. Standard Specifications for Two Course Concrete Street Pavement, 11 pp.
19. Standard Specifications for One Course Concrete Alley Pavement.

The above Standards can be obtained at the price of 25 cents each or in lots of ten or more at 20 cents each. Special price to Members 15 cents or 10 cents each in lots of ten.

BY-LAWS
OF THE
AMERICAN CONCRETE INSTITUTE.

ARTICLE I.

MEMBERS.

SECTION 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of this Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

SEC. 2. A firm or company shall be treated as a single member.

SEC. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

SEC. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener if necessary to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of their qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

SEC. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of the members of the Institute, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In

case of the disability or neglect in the performance of his duty, of any officer of this Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the mettings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and shall furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed

and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

SEC. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

SEC. 2. The annual dues of each member shall be ten dollars ($10.00).

SEC. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for the fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

Article V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such Recommended Practice and Specifications shall be considered adopted unless at least 10% of the total membership shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

Synopses of Recent Articles on Concrete.

Reviewed April 1 to 30, 1915.

The synopses given below are for the purpose of keeping the members of the Institute in touch with the leading articles of interest appearing in the many technical journals and are not complete reviews of these journals. The publications regularly reviewed appear below and the number given corresponds to the number following the title of the paper. The length of the article in pages, whether illustrated or not, is noted, and the publications reviewed in which it appears:

1. Canadian Engineer, Toronto, Ontario, Canada.
2. Cement, New York, N. Y.
3. Cement and Engineering News, Chicago, Ill.
4. Cement Era, Chicago, Ill.
5. Cement World, Chicago, Ill.
6. Concrete Age, Atlanta, Ga.
7. Concrete Era, Los Angeles, Cal.
8. Concrete-Cement Age, Detroit, Mich.
9. Concrete and Constructional Engineering, London, England.
10. Contract Record and Engineering Review, Toronto, Can.
11. Engineering and Contracting, Chicago, Ill.
12. Engineering News, New York, N. Y.
13. Engineering Record, New York, N. Y.
14. Good Roads Magazine, New York, N. Y.
15. Portland Cement, Kansas City, Mo.
16. Rock Products, Chicago, Ill.
17. Proceedings American Railway Engineering Association, Chicago, Ill.
18. Western Contractor, Record Building, Kansas City, Mo.
19. Journal National Fire Protection Association, Boston, Mass.
20. Journal Western Society of Engineers, Chicago, Ill.
21. Proceedings American Society of Civil Engineers, New York, N. Y.
22. Proceedings American Society of Municipal Improvements, New York.
23. Proceedings American Society for Testing Materials, Philadelphia, Pa.
24. Proceedings Engineers' Club of Philadelphia, Philadelphia, Pa.
25. Proceedings Engineers' Society of Western Pennsylvania, Pittsburgh, Pa.
26. Professional Memoirs, Corps of Engrs., U. S. A., Washington, D. C.
27. Proceedings Institution of Civil Engineers, London, England.
28. Transactions, Canadian Society of Civil Engineers, Montreal, Quebec, Canada.
29. Railway Age Gazette, Chicago, Ill.
30. Better Roads, Jamestown, Ohio.

DESIGNS AND TESTS

Long-Span Roof Slab of Concrete and Tile, by Max Miller. (13) Apr. 24, ¾ p., ill. Concrete top surface, large hollow tile, and two-way reinforcement used to span 49 x 46-foot rectangular opening.

Field Methods in Concrete Construction—Column Forms: Design and Construction, by Jerome Cochran. (8) Apr., 3 pp., ill.

Design Methods in Concrete Construction—Analysis of Stresses in T-Beams and Design of Beams in General, by Jerome Cochran. (8) Apr., 3 pp., ill. A mathematical treatise.

BUILDINGS

A Type of Reinforced Concrete Floor for Beam Spans Designed to Replace Wooden Floor. (11) Apr. 7, ½ p., ill.

Concrete-Unit Building Construction at Cedar Rapids. (12) Apr. 8, 2¾ pp., ill. A good example of molding and curing concrete before placing in structure.

Repair of Concrete Buildings at Edison Plant Sets Precedents in Construction Work. (13) Apr. 17, 4 pp., ill. Original estimate of salvage of concrete, 87½ per cent, borne out by data gathered during reconstruction.

Righting the Tilted Grain Elevator of the Canadian Pacific Ry. (8) Apr., 1 p., ill.

Shoring a Large 9-Story Building During Freezing Weather at Detroit, Mich. (8) Apr., 1 p., ill.

HOMES

"In The Woods," a Concrete Home. (7) Apr., 2 pp., ill. Description.

BRIDGES AND RAILROAD WORK

High Concrete Bumping Blocks; B. & O. R. R. (12) Apr. 1, ½ p.

Low-Water Bridges Over Torrential Streams, Bexar Co., Texas, by Terrell Bartlett. (12) Apr. 1, 2½ pp., ill. Designs and descriptions.

New Railway Terminal at Denver, Col. (12) Apr. 1, 2 pp., ill. Showing design of concrete platform shelters of unit construction.

Construction Features of the Reinforced Concrete Cantilever Bridge on Runnymede Ave., Cincinnati, Ohio. (11) Apr. 7, 1¼ pp., ill. Description, details and costs.

Work Involves Expenditure of $10,000,000. Interesting Features. (12) Apr. 8, ½ pp., ill. (1) The high level crossing; (2) the concrete-cased steel bridges; (3) the use of bin cars for the concreting trains and traveling forms for the retaining-walls; (4) the carrying of drainage from the fill to chambers formed in the retaining-walls and having sewer connections; (5) the subway bridges, with steel columns, steel and concrete cross-girders and concrete slab spans.

A Three-Level Railway Crossing at Chicago. (12) Apr. 15, 4 pp., ill. Showing design of piers and abutments and method of encasing steel girders in concrete.

Unexcavated Core Is Left in Center in Sinking Circular Subway Shaft. (13) Apr. 24, 1 p., ill. Saving of 700 yards in material excavated under air. Concrete placed by chute from elevated mixer to which derrick raises dry batches.

Building the Substructure of the G. T. P. Bridge over the Fraser River, Prince George, B. C., by E. Stanley Holland. (10) Apr. 28, 4 pp., ill. Methods used in placing concrete at 35 degrees below zero.

Surface Treatment of Concrete Bridges for Railway and Highway Traffic, by Albert M. Wolf. (8) Apr., 6¼ pp., ill. Description and detail of methods of surface finish employed on several bridges in various parts of the country.

RESERVOIRS, DAMS, CANALS, ETC.

Rock-Fill Dam with Some Extraordinary Foundation Problems, by M. C. Hinderlider. (12) Apr. 8, 4½ pp., ill. Upstream side faced with reinforced concrete. No expansion joints provided.

Data and Discussion on the Design, Construction and Cost of Reinforced Concrete Standpipes, by W. G. Kirchoffer. (11) Apr. 14, 5 pp., ill.

Method and Cost of Constructing Reinforced Concrete Drops, Canadian Pacific Ry., Irrigation Projects, by Robert S. Stockton. (11) Apr. 14, 3¼ pp., ill.

Waterproofing Existing Concrete Structures. (12) Apr. 15, 1 p. Materials used and costs.

Construction Plant and Methods Employed on New Water Works Intake Tunnel at Milwaukee, Wis (11) Apr. 21, 3 pp., ill.

New Tunnel, 400 Feet Below Existing Bore, Replaces Short Section of Damaged Catskill Siphon. (13) Apr. 24, 2⅓ pp., ill. Time limit requires rapid construction at Mooda Siphon in spite of fact that one 8-ft. shaft 400 ft. deep is only means of access to work.

Construction Plant and Methods Employed on New Water Works Intake Tunnel at Milwaukee, Wis. (11) Apr. 28, 4 pp., ill.

Concrete Headgate, South San Joaquin and Oakdale Irrigation District in California. (11) Apr. 28, 2½ pp., ill.

Contract Methods and Equipment for a Typical Ohio River Dam, by Karl H. Shriver. (12) Apr. 29, 6 pp., ill. This article gives in detail the construction methods, the contractors' equipment, and the costs for a typical dam just completed.

Multiple-Arch Dam to Retain Quartz Mill Tailings. (12) Apr. 29, 1½ pp., ill. Description of reinforced concrete dam.

Reinforced Concrete Storage Dam at Swift Current, Sask., Canada, by J. B. Babcock. (8) Apr., 3 pp., ill. Description and design.

Concrete Versus Stone Revetment, in the Kaw Valley Levee Work, by E. B. Murray. (8) Apr. 3 pp., ill.

Notes on Concrete Construction on the New York State Barge Canal, by George C. Mills. (8) Apr., 3 pp., ill.

ROADS AND PAVEMENTS

Comparative Costs of Brick and Concrete Roads in Illinois, by H. E. Bilger. (12) Apr. 1, ¾ p.

Concrete Road with a Single Crack in 4¼ Miles the Result of Careful Construction. (13) Apr. 17, 3½ pp., ill. On Connecticut Turnpike preparation of subgrade, long mixing of each batch of concrete, use of reinforcement, and grading of aggregates were features.

Five Years' Experience with Concrete Pavements at Fredonia, Kansas. (13) Apr. 24, 1¼ p., ill. Long wear and low costs are advantages of utilizing Joplin flint and mine screenings for aggregate in small town work.

Building a Concrete Road in Oregon, by Henry L. Bowlby. (7) Apr., 3 pp., ill. Costs.

Concrete Pavement as Laid at Sioux City, by T. H. Johnson. (7) Apr., 2 pp.

SEWERS, PIPES, TILES, ETC.

Construction Plant and Methods Employed in Building a System of Tunnel Sewers at Edmonton, Alberta. (11) Apr. 21, 4 pp., ill.

Edmonton's Tunnel Sewer System: Features of Design and Construction, by A. J. Latornell. (10) Apr. 28, 6 pp., ill. Constructed of concrete blocks encased in reinforced concrete.

CONCRETE PRODUCTS

The Manufacture of Hollow Concrete Block in the Canal Zone—The Administration Building. (8) Apr., 5 pp., ill. Notes on the manufacture and use of concrete structural tile or block by the United States government in the Canal Zone.

Building Code Regulation of Concrete Block and Other Building Units, by Harvey Whipple. (8) Apr., 3¼ pp., ill.

Notes on Equipment of a Concrete Brick Plant in Hospital Broom Shop, by Harvey E. Jenks. (8) Apr., 3 pp., ill.

MISCELLANEOUS

The United States Lighthouse Service, by George R. Putnam. (12) Apr. 1, 5 pp., ill. Miscellaneous use of concrete in construction of lighthouses.

Weathering of Concrete Prevented by Waterproofing. (13) Apr. 17, ½ p. The Reclamation Service treats vertical surfaces with soap and alum and horizontal surfaces with paraffine.

Municipal Wharves and Sheds at Los Angeles. (12) Apr. 29, 2 pp., ill. Showing construction of reinforced concrete wharf.

Recent Views on Concrete and Reinforced Concrete, by T. A. Watson. (9) Apr., 4 pp. Economy in reinforced concrete construction.

How Design and Methods of Buying Influence the Cost of Steel Reinforcing Bars, by A. D. Mellor. (8) Apr., 2¾ pp.

Details of a Standard Portable Carpenter Shop on Construction Work. (8) Apr., 1 p., ill.

Concrete Superseding Wood in Lighthouse Construction, by James G. McCurdy. (8) Apr., 1 p., ill.

The Building of Concrete Scows is Made a Commercial Success, by Alvin W. King. (7) Apr., 2 pp., ill. Description and illustrations.

Some Elements of Business Success in Concrete Work, by Harry Franklin Porter. (7) Apr., 2 pp. The cost of doing business.

PART II.

PROCEEDINGS OF ELEVENTH ANNUAL CONVENTION.

Papers, Discussions and Reports.

TEST OF A REINFORCED CONCRETE SLAB.

By E. B. McCormick.*

Because of the uncertainty existing among bridge designers and concrete engineers regarding the distribution throughout a concrete slab of the effects of loads, it has been the custom to use a width of from 1 to 4 ft. as a repeating section. It has been recognized that these widths could probably be increased if more were learned regarding the distribution of the stresses. About three years ago the U. S. Office of Public Roads undertook a series of experiments to determine, if possible, the effective width of reinforced concrete slabs under various conditions of loading. The earlier tests were made on slabs 7 by 9 ft. using a span of 6 ft. Some interesting results were obtained, but it was apparent that the size of the slab must be increased if definite, corroborative results were to be secured.

In the summer of 1914, a slab was constructed 12 in. thick by 32 ft. wide, and with a span of 16 ft. The reinforcement amounted to 0.75 per cent and consisted of $\frac{3}{4}$-in. square bars, spaced $7\frac{1}{4}$ in. on centers. The load was applied by means of a hydraulic jack having a capacity of 150 tons. The intensity of the load was measured by the deflection of a chrome nickel alloy steel bar 5 x 7 in. The bar was placed in the line of application of the load. The steel from which the bar was rolled had a tensile strength of 250,000 lb. per sq. in. and an elastic limit of 200,000 lb. per sq. in. The deflection of the steel beam was calibrated on a 200,000-lb. Riehlé testing machine. The apparatus arranged for applying a four-point load is shown in Fig. 1.

In this first test no attempt was made to determine the effect of cross reinforcing, the slippage of the steel, or the fibre stresses along the bottom of the slab. The slab was designed for a fibre stress of 650 lb. per sq. in. along the top surface. Cylinders 8 x 16 in. cast from the same mix showed a modulus of elasticity of 2,900,000 at an age of 6 weeks.

* Mechanical Engineer, Office of Public Roads, U. S. Department of Agriculture, Washington, D. C.

The deformations along the top surface were measured at 20-in. intervals by means of a Berry strain gage. From these deformations the fibre stresses were calculated by the formula: $x = dE = RCE$, where R is the reading of the dial, and C the constant of the strain gage, in this case 0.01.

The first readings were taken with center loads of 25,000, 30,000, and 32,500 lb. per sq. in. In Fig. 2 are shown the lines of equal stress resulting from a center load of 30,000 lb. The cross-hatched portion of this diagram shows the intensity of the

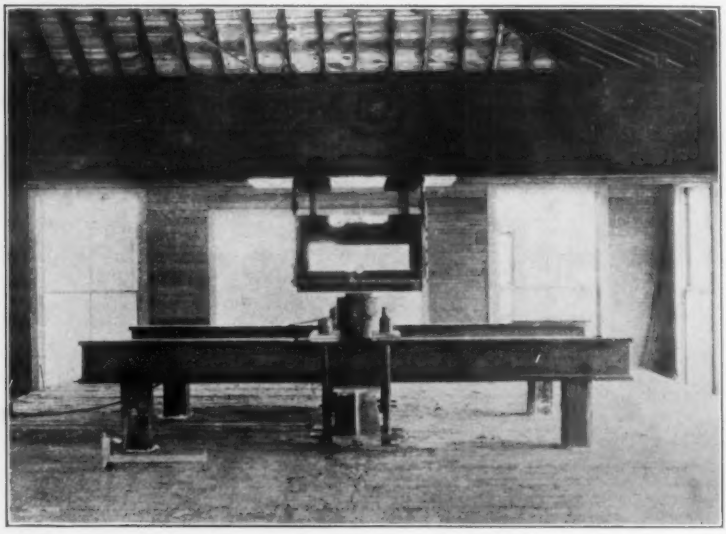


FIG. 1.—APPARATUS FOR APPLYING FOUR-POINT LOADS.

top fibre stress along a section passed through the point of application of the load and parallel to the supports.

The effective width of slab under each of the three loads is shown on Fig. 3. This "effective width" is considered to be the width of a slab, which, if subjected to a load of a certain magnitude, uniformly distributed along the top fibre of the center section, will have the same maximum outside fibre stress and the same total fibre stress as the tested slab showed under the concentrated load of that magnitude. It will be noted from the diagrams in Fig. 3 that the effective width is the same for loads of

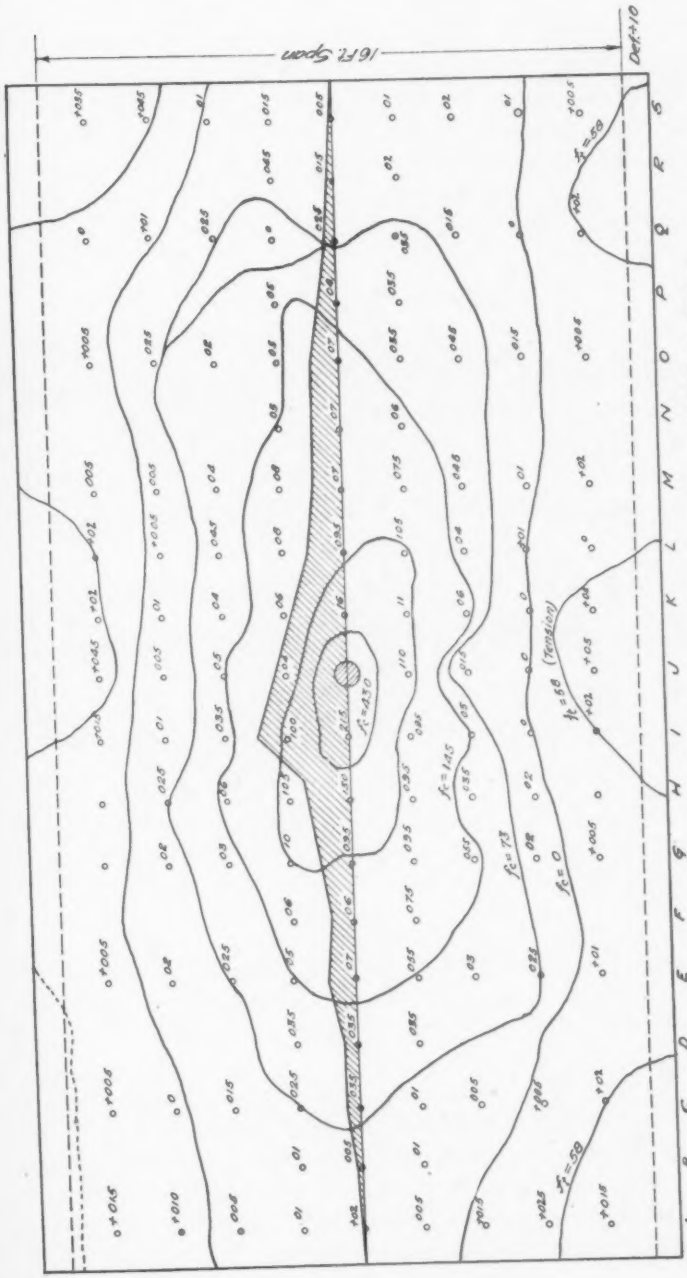


FIG. 2.—LINES OF EQUAL STRESS. CENTRAL LOADING OF 30,000 LB. TOTAL.

different magnitudes; *i. e.*, the effective width of a slab is apparently independent of the magnitude of the load.

Fig. 4 shows the line of equal stress, or equal deformation, for a two-point load of 80,000 lb.; and the upper and lower diagrams on Fig. 5, the fibre stresses along the center section parallel to the support for two-point loads of 40,000 lb. and 80,000 lb., respectively. This two-point load was applied along the center

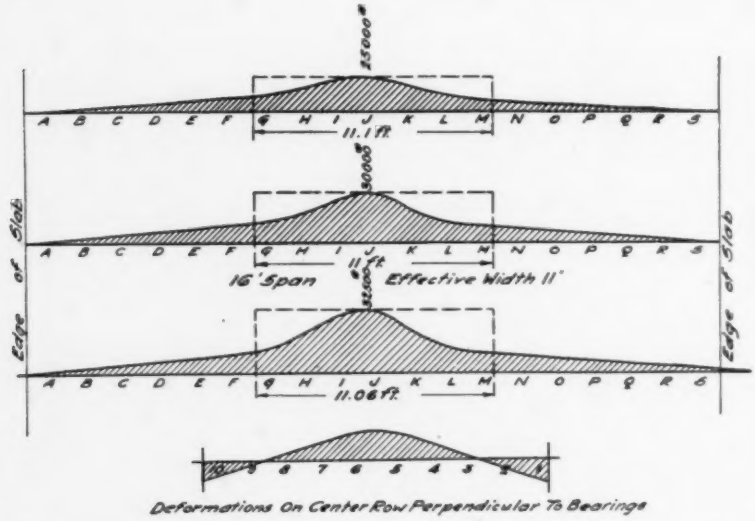

25000#
A B C D E F G H I J K L M N O P Q R S
11.1 ft.
30000#
Edge of Slab
A B C D E F G H I J K L M N O P Q R S
11 ft.
16' Span
Effective Width 11'
32500#
Edge of Slab
A B C D E F G H I J K L M N O P Q R S
11.06 ft.
Deformations On Center Row Perpendicular To Bearings

FIG. 3.—DEFORMATIONS ON CENTER LINE OF SLAB NO. 836. SPAN, 16 FT.; WIDTH, 30 FT.; EFFECTIVE WIDTH, 11 FT.; EFFECTIVE DEPTH, 10.5 IN. (CONCENTRATED LOADS OF 25,000, 30,000 AND 32,500 LB. CENTER LOADING).

line parallel to the supports, and equidistant from the center of width of the slab. The points of application were 6 ft. apart. The maximum total load was 80,000 lb. or 40,000 lb. on each point.

In the hope of securing conditions somewhat similar to those met with in the case of a bridge carrying a traction engine, road roller, or loaded truck, a four-point load was applied, as shown in Fig. 1. The total maximum load in this case was 100,000 lb. The points were arranged symmetrically about the center, 8 ft. apart longitudinally, and 5 ft. transversely. The equal stress line

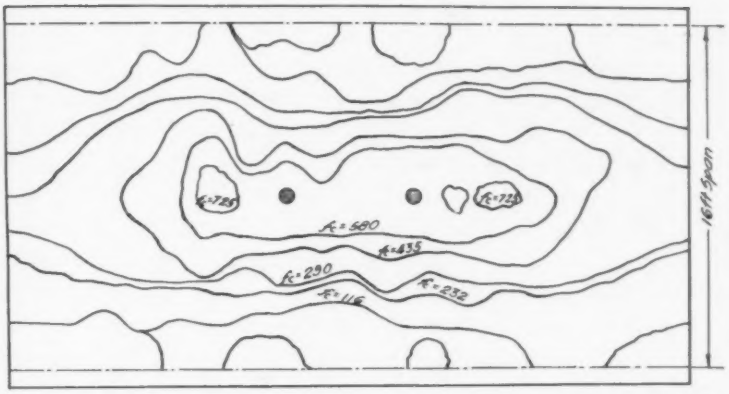


FIG. 4.—EQUAL STRESS LINES ON SLAB NO. 835. TOTAL LOAD OF 80,000 LB. ON TWO POINTS.

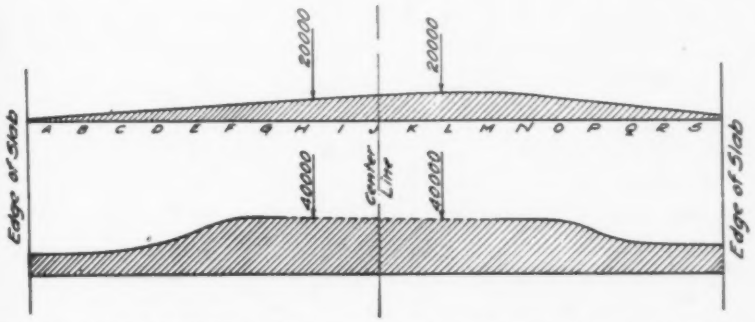


FIG. 5.—FIBRE STRESSES ALONG CENTER SECTION FOR TWO-POINT LOADINGS.

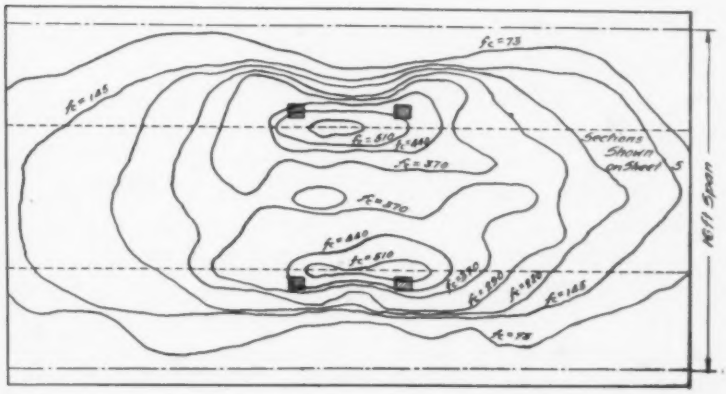


FIG. 6.—EQUAL STRESS LINES ON SLAB NO. 835. TOTAL LOAD 100,000 LB. ON FOUR POINTS.

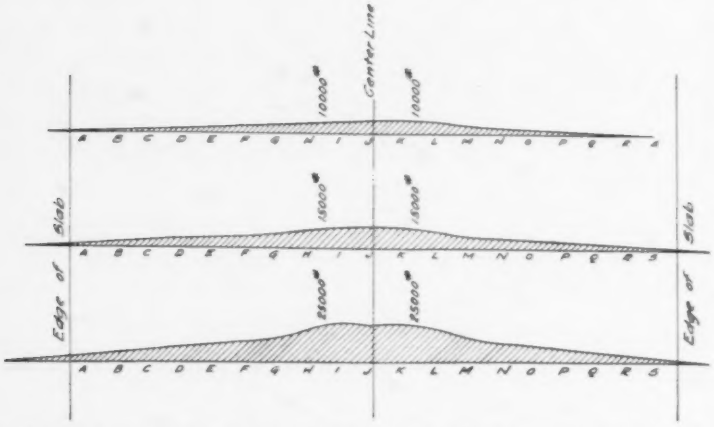


FIG. 7.—DEFORMATIONS OF CONCRETE ALONG DOTTED LINES AS SHOWN ON FIG. 6. AVERAGE OF 3–4 AND 7–8 ARE PLOTTED. TOTAL LOAD ON SLAB 100,000 LB. ON FOUR POINTS.

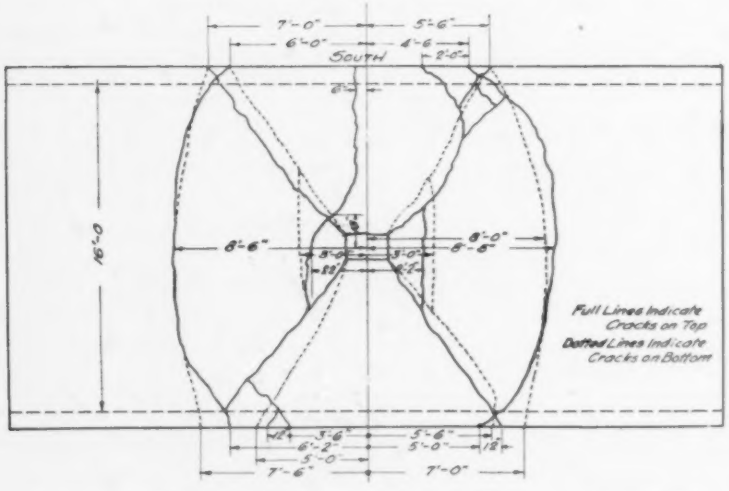


FIG. 8.—LOCATION OF CRACKS IN SLAB NO. 835 WHEN TESTED TO DESTRUCTION. TOTAL LOAD 119,000 LB. ON CENTRAL PLATE 12 x 24 IN.

diagram for this load is shown in Fig. 6, and the top fibre stresses in the lower diagram in Fig. 7. The upper and middle diagrams in this figure show the fibre stresses for total loads of 40,000 lb. and 60,000 lb., respectively.

On November 16, 1914, this slab was tested to destruction in the presence of some fifty visiting engineers. The load was applied at the center on a plate 12 x 24 in. The object in using this plate instead of the 6-in. cylinder used in previous tests was to avoid punching a hole through the slab. The load was applied steadily until failure, observations of cracks and other phenomena being noted as work progressed. A total load of 119,000 lb. was applied. Failure was by a combination of tension in the bottom and compression in the top fibres. The width of slab over which failure extended was about 17 ft. The location of the cracks is shown in Fig. 8. The full lines show those formed on the top; the dotted lines, those on the bottom. It will be observed that these cracks are contained within an approximate circle of 17 ft. diameter.

The data obtained from the tests of one slab do not, of course, warrant the drawing of final conclusions, but they are of particular interest in that they confirm the results indicated in the tests of the smaller slabs.

It is the intention to continue the work until a sufficient number of slabs have been tested to warrant the deduction of definite laws relating to the distribution of stresses in loaded concrete slabs. There is now under construction a thinner slab of the same area, per cent of reinforcement and proportions of materials. The tests on this slab will include measurements to determine fibre stresses on the bottom of the slab and also the slippage of the reinforcement. The object in using a thinner slab for the second test is to determine, if possible, what effect the thickness of the slab has on the effective width.

Arrangements are being made to secure the co-operation of interested engineers, through an advisory committee that is to be appointed to confer with the engineers of the Office of Public Roads in laying out the work to be done in the future. It is hoped that colleges and experiment stations will carry on series of tests of this nature, the results of which may be correlated with those obtained by the Office of Public Roads.

DISCUSSION.

Mr. Abrams. MR. DUFF A. ABRAMS.—I would like to ask how the measured stresses compared with the computed stresses.

Mr. McCormick. MR. E. B. MCCORMICK.—We measured the deformations, and from those measurements computed the stresses in the fibers. We had no means of measuring the stresses except from the deformation of the fibers.

Mr. Abrams. MR. ABRAMS.—What I had in mind more particularly was an increase of the stresses you used in the design, for instance.

Mr. McCormick. MR. MCCORMICK.—We exceeded those. The design was based on the expectation of a fiber stress of about 650 lbs. We had in some cases, I think, as high as 800 lbs. The 2-point load of 80,000 lbs. was larger than we intended to use.

Mr. Goldbeck. MR. A. T. GOLDBECK.—There is one peculiarity about the test which might be mentioned. The effective width obtained by Mr. McCormick was about 11 ft., as I recall it; and the span length was 16 ft. The effective width is approximately seven-tenths of the span length. Now, suppose we substitute in our ordinary concrete beam formula the fact that the effective width varies directly as the span length, we will find that the carrying capacity of the slab is independent of the span length. I am greatly in doubt whether this relation will hold for all conditions. As I remember it, the effective width in the preliminary tests was independent of the lateral reinforcement; and that is reasonable. The shearing strength of the concrete is sufficient to transmit the stresses from the center out to the sides without any aid from lateral reinforcement. This is no argument, however, for the omission of lateral reinforcement.

Mr. McCormick. MR. MCCORMICK.—In connection with the point that Mr. Goldbeck has just brought out about the length of the slab, I will say that we made some tests after we broke this slab, taking small slabs $2\frac{3}{4}$ in. thick, made of one part cement and one part sand, and tested them on about 10 or 12 in. centers. These slabs varied in width, to probably 30 in. We found in the case of the slabs in which the width was not greater than the span, that we

had a curve running up at approximately 60 deg., and very close to a straight line. After we got beyond that point the curve flattened out and we do not know what happened. In fact, that is what we are now trying to determine. We do not know the exact relation, but it is apparent that the ratio of the length to the span is going to enter in accordance with some law. Mr. McCormick.

MR. GOLDBECK.—There is another point, which might be brought out: If you remember, the width over which the cracking took place in the slab just reported was about equal to the span length. The width over which the cracks extended was 17 ft., whereas the span length was 16 ft. A corresponding result was obtained in the preliminary tests mentioned by Mr. McCormick. Similar tests have been made by the Ohio State Highway Commission, and also by the University of Illinois. The effective width obtained in the preliminary tests varies from about $\frac{8}{10}$ of the span length, up to a width approximately equal to the span length. There are any number of problems of this kind that are in a rather indefinite state. For instance, one of them is the problem of the distribution of a concentrated load over the floor beams of a bridge floor of concrete jack-arch construction. We have a problem of that kind under investigation and have already tested two bridge floors by piling weights on a specially designed loading platform. The indications are that the maximum stress in the lower flange of the I-beams adjacent to the I-beam upon which the load is placed, is about $\frac{1}{2}$ the stress in the center I-beam; and I have not the slightest doubt that the stress extends out even farther than the I-beam adjacent to the center beam. Mr. Goldbeck.

MR. WILLIS WHITED.—I hope this work will be carried on until definite results are obtained. It is a good idea to put on 4-point loads, 2-point loads, etc., but I think that much more information can be obtained by putting on 1-point loads, because if a 2-point load is applied the stress shown at any particular point is due partly to both of them and it is impossible to say how much is due to each. The results are of no value in determining the distribution of stress for different loadings, also, the position of the cracks is not much of a criterion in determining the distribution of stress. I place much more dependence on the elastic diagrams, for the simple reason that the final rupture would Mr. Whited.

Mr. Whited. occur about the shearing strength of the concrete, which would vary considerable according to the amount and position of the reinforcement, whereas, the elastic deflection would be nearly independent of the reinforcement. I do not see any necessity for making a very great number of tests that way, except for the reason that concrete is a very variable material, and naturally the results on different slabs of the same dimensions will differ widely, and it is necessary to make a number of tests and average them, to get reliable results.

Mr. Tubesing. MR. W. F. TUBESING.—The stress in a structure should be investigated with a view to economy in construction. Spans should be reinforced in two directions; consequently, if these tests were made with the stress in one direction only, it would be far from economical construction, because in highway bridges two side beams act as railings, and at the same time carry part of the load. Therefore, the longitudinal steel will carry one portion of the load, and the transverse steel will carry the rest of it. Under these tests results in one and not in two directions are obtained.

Mr. McCormick. MR. MCCORMICK.—The object of making the test with the reinforcing in but one direction first, was to secure information relative to the concrete itself; not to complicate the results with cross reinforcing, but to get information of the distribution of the stress throughout the concrete. Later on in the series we expect to use cross reinforcing. We want to be sure that the results, if we get results, are due to one thing only, and then from time to time add other variables. That was the object in making this first test; and the reason for not attempting to get the slippage of the steel or the stresses on the bottom of the structure.

Mr. Whited. MR. WHITED.—I would like to suggest that some tests be made of the distribution of load over stringers connected by a continuous slab, as was suggested here. It is an important matter, and the mathematical solution is extremely complicated.

THE DESIGN OF CONCRETE HIGHWAY BRIDGES, WITH REFERENCE TO STANDARDIZATION.

By C. B. McCullough.*

No bridge structure can ever be safely and economically designed, nor should the preparation of such design ever be attempted without an accurate and adequate knowledge of the behavior at all times and under all conditions of the waterway over which it is desired to place a bridge. In treating the question of bridge design, therefore, a study of stream conditions logically assumes first place in point of time and may be so placed in order of importance, particularly if the construction is of concrete, inasmuch as this material when once misplaced can be corrected only at a tremendous expenditure, and if the expedient of wrecking and rebuilding must be resorted to the labor cost is always excessive and practically none of the material can be salvaged.

Stream Behavior and Its Effect on Design.

The geologist has classified the three stages of stream development as the stage of youth, the stage of maturity and the stage of old age. The first stage begins with the formation of the first gulley on the hillside and continues until the stream has established for itself a grade or base level. Throughout this period the stream is eroding its channel, but has done no work as yet on its flood plane. In the younger streams in this stage the waterways are carried in comparatively shallow channels and the headroom available in bridging consequently low. The end of this stage, however, finds the stream channels considerably deeper.

The second stage of stream development witnesses the cutting and formation of the flood plane but no further erosion of the channel. The general gradient now being more uniform, as the general valley or flood plane is widened the stream enters the "meandering" stage and the danger of a channel change is in-

* Designing Engineer, Iowa State Highway Commission, Ames, Ia.

creased. The water at ordinary stages is now carried in deeply cut ravines and headroom is in general no longer limited.

The third stage of development witnesses a complete erosion of the flood plane and valley. The streams in this stage are in general filling rather than cutting, and are even more unstable or meandering than in the preceding stage.

The first or youthful stream is found in the newer drift areas of the glaciated region which occupies the northern part of the United States and in the mountainous regions of the unglaciated area. The second and third stages are encountered in the older glacial drift areas (The Kansan and the Pre-Kansan) and in the unglaciated lowlands. The topography accompanying these latter stages of development being older, a more complete weathering and breaking down of the surface materials has taken place and in general concrete materials are encountered less frequently than in the former areas.

The stage of stream development has a close bearing on that most important consideration of design, the selection of type, thus for the first or youthful stream the low available headroom ofttimes renders the selection of the through girder or flat slab type of construction imperative, while the low pier heights may throw the economy on the side of short multiple span construction. A plentiful deposit of glacial gravel near the bridge is not unusual with this topography and may so cheapen the hauling cost as to make the mass type of construction cheaper than the relatively light, heavily reinforced types.

For streams in the second and third stage of development, the channels being now completely cut are in general deeper and the saving of headroom is ofttimes a minor consideration. A frequently encountered scarcity of concrete materials and the resultant long haul for country highway bridge work renders massive construction in general more expensive than the heavily reinforced type.

The greatest difference in behavior of streams in the three stages of development, and the one that most vitally effects bridge design, is the difference in the eroding qualities (Fig. 1). While the cutting power of a stream varies as the square of the velocity, the carrying capacity varies as the sixth power of the same. The eroding power which lies between these two values may in alluvial

soils assume a value very near the latter figure as the question becomes practically one of carrying capacity. The results of this increased eroding power during periods of high water and consequent increased velocity are truly appalling in magnitude. This quality in youthful streams necessitates extra precaution as regards scour, such as aprons, cross walls, paving of stream beds, etc., and makes it imperative that all foundations shall be carried well below ultimate scouring lines and be protected in every way against erosion.

FIG. 1.—THE EFFECT OF EROSION.

Streams in the more mature stages, having lost the power of erosion and having in general well-defined banks and broad flat flood planes, can be bridged ofttimes with the pedestal or viaduct type of substructure in entire safety. The stream is now in its meandering stage and the instability of the stream in its banks must be taken into consideration and guarded against, although in general no new channel will erode below the established base level of the stream.

The development of the stream may be arrested at any time by artificial conditions and made to assume characteristics of an

entirely different stage of development, thus the straightening of the channel of the Nishna Botna River in Pottawattamie County, Iowa (a mature filling stream), has caused the stream to renew its youth with a consequent erosion that has proven disastrous to several bridge structures.

In concluding the subject of stream behavior it must be pointed out that certain of the conditions affecting selection of type, such as available headroom, accessibility of concrete materials, etc., are largely local and must be worked out for each individual

FIG. 2.—THE EFFECT OF FOUNDATION SETTLEMENT.

structure, but that their relationship to stream development and topography is extremely important in fixing the type chosen for standard construction for small work over any considerable area.

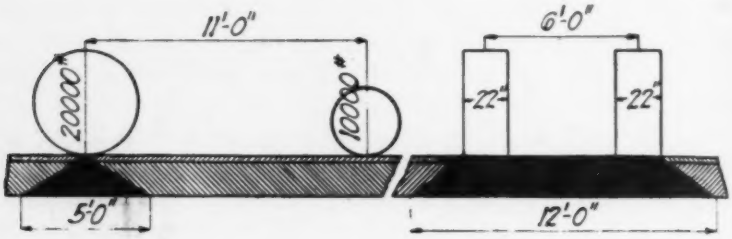
The question of stream erosion cannot be too carefuly studied. Probably 90 per cent of our concrete bridge failures are foundation failures (Fig. 2) and by far the majority of these are due to erosion of foundations rather than to insufficient bearing. Bearing, either natural or artificial (piling), can be secured if careful soundings are made (which, of course, should in every case be done), but

protection against erosion requires a careful study of stream conditions. The tendency of any stream, if the materials along its bed are equally resistant to abrasion, is to establish for itself a uniform grade, and thus if for a short distance either side of a proposed bridge site a profile of the present stream bed is secured, a grade line platted thereon will furnish valuable information as to the probable future base level or grade of stream. The necessity for a study of this kind as a part of each bridge survey cannot be too strongly emphasized.

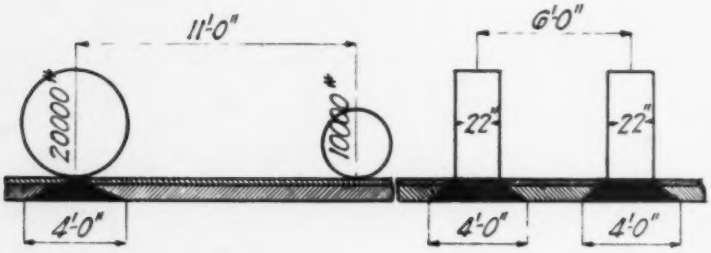
LOADINGS.

Next in the order of time and importance comes the question of loadings. The actual weight of the dead and live load is, of course, a matter of unit weights of material, such as filling, paving, etc., and of the weights of the maximum artificial loads to which the structure based upon past experience and future traffic requirements is likely to be subjected. The determination of the distribution of load concentrations and the amount and effect of internal temperature variations constitute an equally important, albeit a much more complicated problem. The distribution of load concentrations has of late been the subject of considerable experimentation, although no great amount of data has as yet been made public. Such a series of experiments are now under way by the Engineering Experiment Station of the Iowa State College and the State Highway Commission of Iowa. Two full size test spans, one a reinforced concrete slab and the other an I-beam span with reinforced concrete floor have been constructed for this purpose. The external loading is applied by means of a pair of hydraulic jacks of thirty tons' capacity and the distortions both in concrete and reinforcing steel measured direct by a Berry strain gage with an Ames dial. While the results of these tests are as yet far too incomplete for presentation, as far as they have gone they seem to indicate that the assumptions usually made are safe.

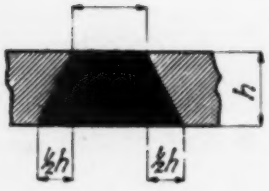
The total load distribution may be considered as composed of two portions; that due to the strength of the section itself and that due to the fill or covering material. The assumption as to load distribution due to both of these factors, for the standard traction engine loading adopted by the State Highway Com-



Loading Distribution for 15 Ton Traction Engine on Slabs 12" or more Thick.



Loading Distribution for 15 Ton Traction Engine on Slabs less than 12" Thick.



Distribution of loading thru earth fills of 12" or more. Thru fills of less than 12" no distribution is assumed.

Iowa Highway Commission
Standard Loading Distribution for 15 Ton Traction Engine
1915

FIG. 3.

mission of Iowa, is shown by Fig. 3. This may be said to represent average present-day engineering practice for highway loadings.

The determination of the internal temperature range in concrete, heretofore a matter of much conjecture, was during the years 1908 to 1912 made the subject of a series of experiments by the Iowa State College Engineering Experiment Station and the Iowa State Highway Commission. These tests were made on two reinforced concrete arch structures, one located on Walnut Street over the Des Moines River at Des Moines, Iowa, and the other on Boone Street over Squaw Creek at Ames, Iowa. Two methods of measurement were used, one that of the mercurial soil thermometer imbedded in the concrete and the other that of electric resistance coils imbedded in the concrete and read by means of the portable pyrometer of the Thwing type. Daily readings were taken for a period of about three years and temperature time curves platted as shown in Fig. 4, which is a typical curve for bulbs No. 2, No. 6 and No. 8 in the Walnut Street Bridge, from August, 1911, to January, 1912.

Based on the results of these experiments the following conclusions were stated:

1. The yearly range in temperature in a reinforced concrete arch structure typical of the highway arch construction in Iowa is, in this latitude, Iowa, not far from 80 deg. Fahrenheit.

2. The relation between the depth of concrete covering at any point and the yearly temperature range may be obtained from the curve of Fig. 5, which is plotted for the results obtained on these particular bridges.

The results of the foregoing investigation lead to stress values many times in excess of those which have heretofore been assumed to result from temperature variation and have considerably modified the shape and size of the concrete highway arch as adopted in Iowa. This will be set forth in detail further on

DETAILS OF DESIGN.

Now comes the calculation of stresses and the working out of details. While space will not permit a complete discussion of the various methods followed in highway bridge design, a brief discussion of a few of the more common types of concrete struc-

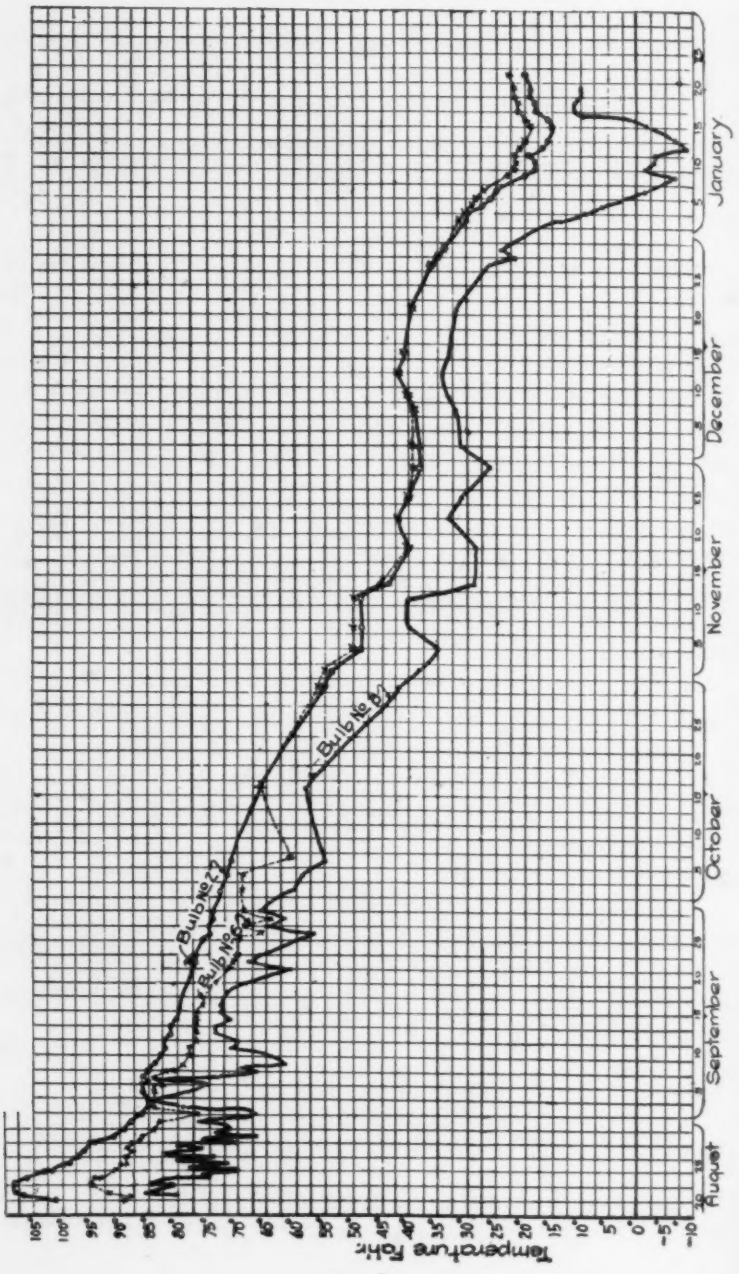

Temperature Fahr.
105° 100° 95° 90° 85° 80° 75° 70° 65° 60° 55° 50° 45° 40° 35° 30° 25° 20° 15° 10° 5° 0° -5° -10°
August
September
October
November
December
January
Bulb No. B1
Bulb No. B2
Bulb No. B3

FIG. 4.

tures adopted as standard by the State Highway Commission of Iowa may be of interest as illustrative of the recent development along this line.

Superstructures.—The slab type, as the name would imply, consists of a simple beam or slab, reinforced for main and diagonal tension by longitudinal rods, and for lateral distribution, shrinkage and temperature stresses by transverse rods.

The deck girder type (Fig. 6) depends for its economy on the fact that the carrying capacity of a beam varies as the square of the depth, but only as the first power of the width. Thus the use of deep narrow girders operates to reduce the amount of material

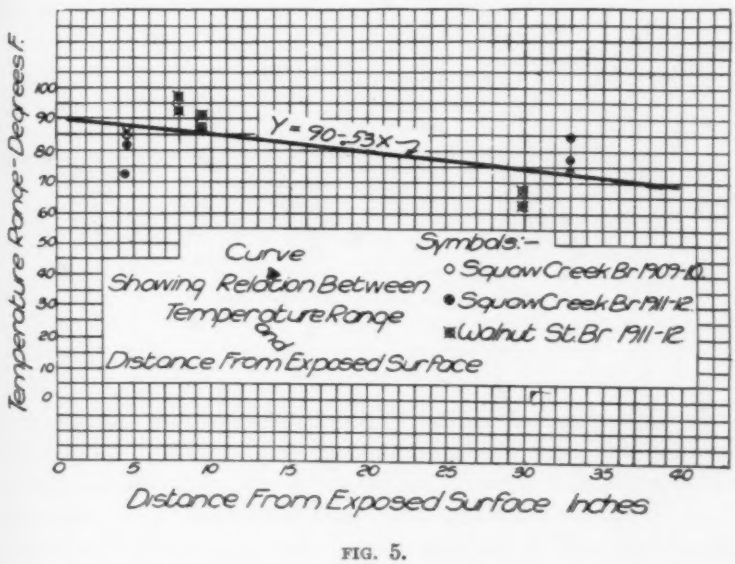


FIG. 5.

and consequently cut down the cost. The stresses arising in the superstructure are transmitted into the girders by means of a thin doubly reinforced floor which acts as a beam partially fixed and partially continuous. The girders themselves are reinforced for main and diagonal tension both by bent-up rods and by stirrups.

In the through type (Fig. 7) of girder construction two main side girders support the floor, which is hung or suspended between

them. The main girders act as simple beams and the floor as a partially fixed beam, the point of contraflexure being taken at about 12 in. from the girder, this dist. ce being based upon experiment. A comparison of quantities for the three types of superstructure is graphically shown in Fig. 8. The economy of cost of the deck type over the through type is probably represented by the relative position of the quantity curves, the form work being equally difficult for both types. The cost of slab construction is, on the other hand, somewhat cheaper than the quantities would indicate on account of the simple form work, and this type of construction is probably the economical one for spans below 20 or 22 ft. The economy of the through girder and the slab above this latter span is one of headroom rather than of cost.

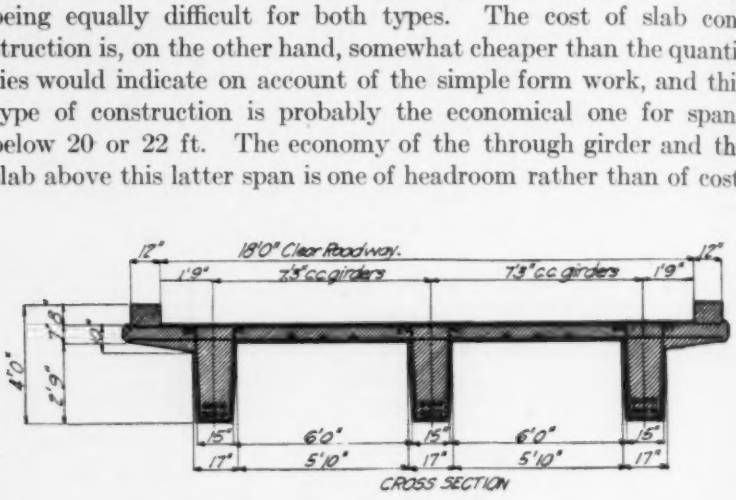


FIG 6.—DECK TYPE GIRDER CONSTRUCTION.

Substructure.—The first type of concrete substructure construction was patterned after that of stone masonry, being heavy and massive and depending for its stability on the weight rather than the structural integrity of the material. From this type there evolved the cantilevered or reinforced stem, whose stability depended upon the weight not only of the structure itself but also of the earth filling placed over the cantilevered base. Considerable economy resulted in the use of this type of construction wherein reinforcing steel in tension was made to replace weight heretofore furnished only at the expense of extra material.

A distinct step in advance of this latter type is the so-called "unit" type, wherein the wing walls are made to give added stability to the entire construction. This is accomplished by securely anchoring the wings to the abutment body by means of

tension rods so placed that in overturning the entire wing must be lifted together with abutment about the point of rotation. The rigid anchorage of the wing produces a movement of the center of gravity backward, or away from the stream, and a consequent lowering of the earth pressure. The amount of this decrease in pressure is graphically shown in Fig. 9. The entire unit action is dependent upon the tension rods which are proportioned to lift the entire wing wall at the beginning of rotation.

The safeguarding and beautification of the approach is an important factor in bridge design and one to which by far too little attention has been paid in the past. Effusive ornamentation of the structure itself has ofttimes been apparently thought sufficient to compensate for a deplorable lack in this respect. As a simple,

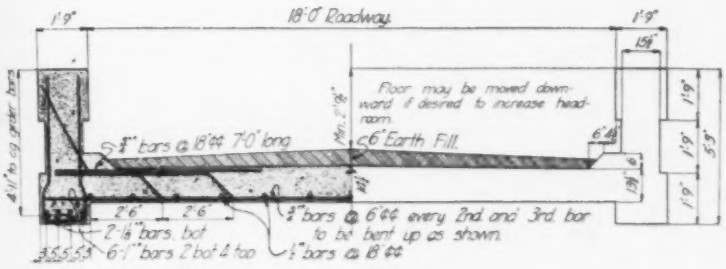


FIG. 7.—THROUGH TYPE GIRDER CONSTRUCTION.

yet effective method of producing a pleasing approach effect and at the same time safeguarding against accident due to the sudden narrowing of the roadway at the bridge site (a condition that for reasons of economy will continue to exist for some time), the wing walls on the standard short span construction have been carried level, out to a point sufficient to give a clear distance between them equal to the width of the traveled way and handrails constructed thereon. The effect is simple, yet giving a finished appearance not otherwise produced. (Figs. 10 and 11.)

No discussion of highway bridge design is complete without a treatment of the fixed arch of reinforced concrete. From an architectural standpoint the arch has been and always will be a favorite because of the quiet, simple dignity of its lines. The stresses arising in the ring of an arch are indeterminate statically

and can be computed only from the theory of elasticity. They are thus seen to be dependent not only upon the loadings but upon the size and shape of the ring as well.

The working out of the true economical shape and section for the arch type is a problem considerably involved and concerning which a great deal has been written in the past. The recent data collected as to temperature variation, however, has operated to modify to a considerable degree the limitations of design and the economical section, and much of the data published is now of little value, especially for the lighter highway loadings. This is shown by a consideration of the following facts.

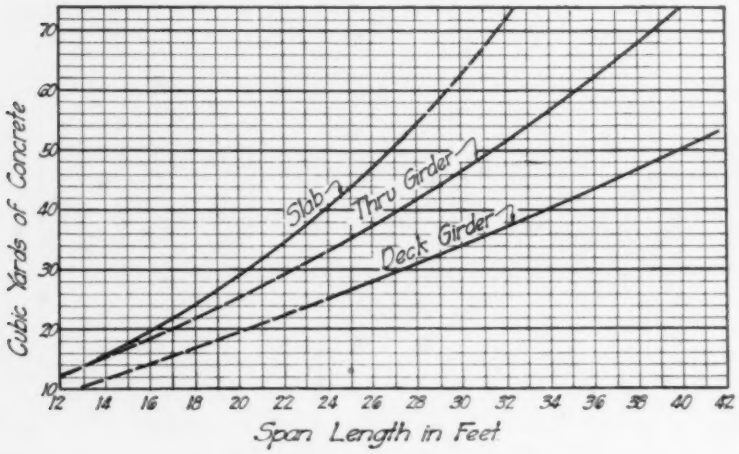


FIG. 8.

In the design of the concrete arch it has been customary among many designers and engineers to allow an arbitrary percentage (from 10 to 20 per cent) of the dead and live load stresses to cover stresses induced by temperature change. All stress values then become a function of the external loading and the following relationships can be easily shown to obtain:

1. For a given span the moments are independent of the rise, but the thrusts vary inversely as the same.

2. The fiber stress being largely determined by the bending moment rather than the thrust does not increase rapidly with a

decrease in rise and analyses of very flat arches ofttimes fail to indicate serious overstress.

3. The unit fiber stresses vary approximately as the bending moment and inversely as the square of the depth of section. Thus a condition of overstress can be remedied by simply increasing the section as the moments and thrusts remain practically unchanged.

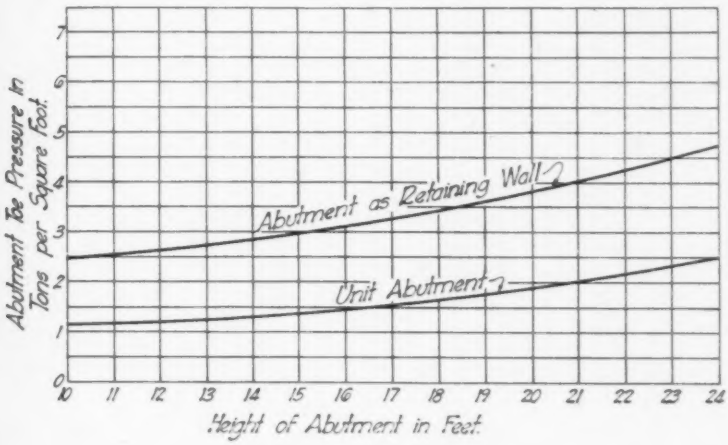


FIG. 9.

The tests conducted on the Squaw Creek and Walnut Street bridges disclose a temperature condition far in excess of that formerly believed to exist, and the moments resulting are found to be nearer 100 per cent than 10 per cent of the dead and live load values as is illustrated by a comparison for five typical arch spans given in the following table:

Arch Ring.	Span.	Bending Moment.		Live Loadings.
		D. & L. Load.	Temperature.	
A	40	8,466	30,302	U. L. L. 100 lbs. per sq. ft.
B	55	34,050	43,800	U. L. L. 200 lbs. per sq. ft.
C	60	30,791	38,310	60 Ton Elec. Eng.
D	70	48,676	45,700	40 Ton Elec. Eng.
E*	90	23,350	31,580	U. L. L. 100 lbs. per sq. ft.
F	70	71,724	50,300	60 Ton Elec. Eng.

* Light open spandrel construction.

In view of the fact that the temperature variation, particularly for light highway loadings, becomes the determining factor the

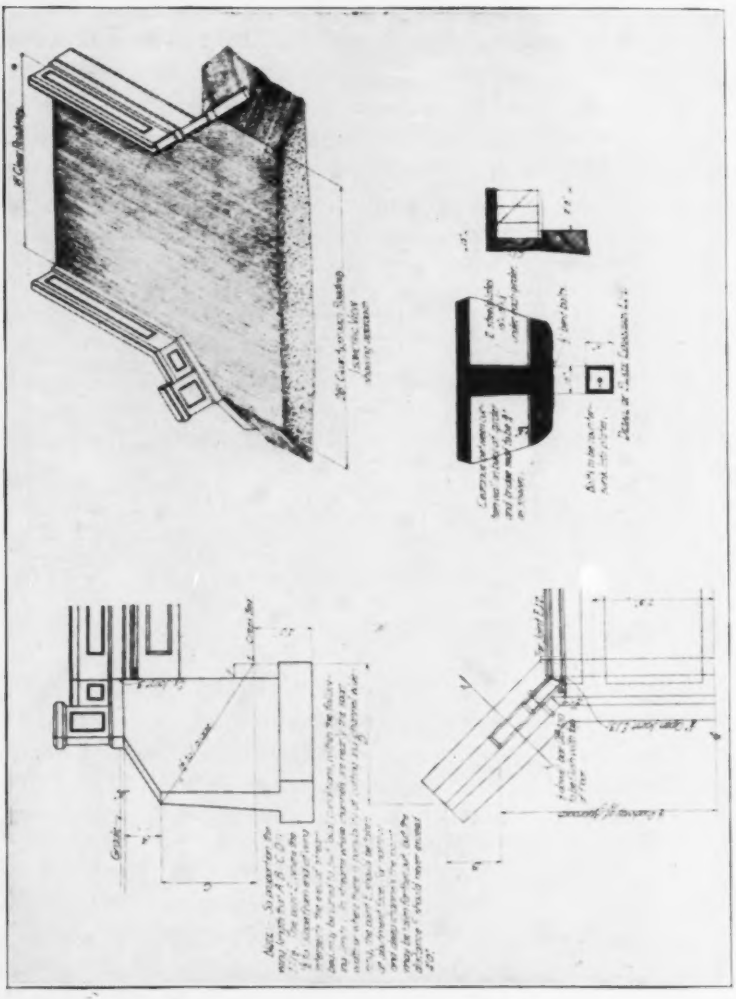


FIG. 10.

following relationships which can easily be proven seem to have an important bearing on the limitation of the design.

1. For a given span the *moment due* to *temperature* varies

inversely as the rise and the corresponding thrust varies inversely as the second power of the same. Thus a slight decrease in rise will produce a material increase in ring stress.

2. While the fiber stresses for a given moment vary *inversely* as the *square of the depth* of section, it must be remembered that in

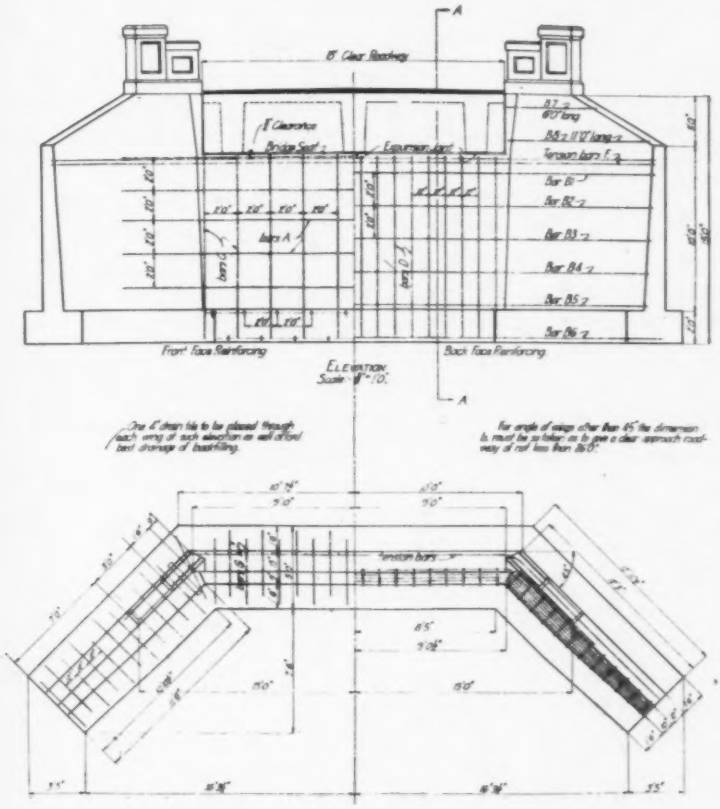


FIG. 11.

dealing with temperature changes the moments themselves vary *directly* as the *cube* of *the depth of section.* Thus when the temperature moment constitutes the larger part of the total moment an increase in section ofttimes operates to increase rather than decrease the unit fiber stresses.

3. The stresses due to temperature are maximum at the crown and spring line of the arch but of opposite sign. These decrease from crown to spring line, passing through a zero value, and assume their original or crown values at a point near the quarter (this point can be determined exactly for every arch, it being dependent upon the shape and size of the arch ring).

From the first two considerations it is at once apparent that for each span length there will be a limiting minimum rise beyond which it is not safe to design. From the third consideration it is apparent that for light highway loadings in which temperature stresses are largely governing there need be in general but slight increase in section between the crown and quarter point.

Thus far the discussion has been limited to superstructure design. For the substructure or abutment, inasmuch as the entire theory of stress distribution in a fixed arch is based upon a hypothesis of fixed and immovable abutments, great care should be exercised to insure such rigidity of base. Construction of the arch type over youthful streams is extremely hazardous unless solid rock is encountered and in any location the greatest precaution to secure adequate foundation bearings is absolutely essential to the integrity of the superstructure.

In concluding the subject of arch bridges it should be pointed out that by far too little information is available concerning the action of the arch. The larger portion of those small cracks, the elimination of which constitute such a problem to the concrete bridge builder, occur in the arch type. Very few arches are economically designed and the great need of experimentation along this line is apparent to all who are familiar with the problems presenting themselves to the designer.

The State Highway Commission of Iowa is endeavoring to meet this need by a series of experiments based upon the results of which it is hoped to present a series of standard arch superstructures designed with a true regard for safety and economy.

REPORT OF THE COMMITTEE ON STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES AND CULVERTS.

Your committee submits the accompanying revision of the progress report published in January, 1914, Journal and presented at the last Convention for the purpose of inviting criticisms and suggestions, and it was the hope of the committee to be able to supplement this report at this Convention.

The work of this committee is not by any means complete. As it develops, new problems arise, many of them of great importance. A high State official made the prediction in a public address that within the next twenty-five years the people of the United States would spend twenty-five thousand million dollars for road construction. Now, a very considerable proportion of this will go for bridges; and, judging from the above prediction, the people of the United States, in the time mentioned, could easily lose more than one billion dollars by unwise specifications for their bridges. I therefore bespeak the earnest co-operation of the engineering profession in the work of this committee.

The aim of the committee is, by co-operation with the engineering profession and others interested, to formulate a system of rules and principles which shall have the approval of the engineering profession and of others best able to form opinions on these subjects to the end that the safety and convenience of the traveling public may be provided for, and also, that public funds may be conserved.

Other questions, which involve very large sums of money and which, so far, do not appear to be solved in a satisfactory manner, may be mentioned as follows:

The distribution of load through an earth fill has been the subject of some investigation, and at the request of the Pennsylvania State Highway Department a series of tests have been

made at State College, Pa.,* to obtain data looking to its solution. Thus far no adequate theoretical investigation of this subject appears to have been made. It is hoped that this can be done in the near future.

The lateral distribution of loads in reinforced concrete slabs is under investigation by the office of Public roads at Washington, D. C. Valuable information should be forthcoming from that source before long. It is hoped that the lateral distribution of load over ribbed slabs may be taken up soon. Some work is being published on this subject but it is widely scattered, and the mathematics involved is very difficult. The committee hopes that some member or members of the Institute will prepare a full discussion of this subject, reducing it to a form which will be of practical use to the profession. The loading imposed on a bridge by a high fill is very important and the committee would appreciate greatly the results of any experience or experiment forwarded to them from any source. The proper allowance for variations of temperature in the design of hingeless arches has been discussed by Mr. McCullough but this subject should be pursued still further.

The subject of the bearing power of soils has been taken up by a committee of the American Society of Civil Engineers. It is hoped that the results of their studies will be very valuable.

The subject of pile and pile driving is very important and the committee desires further information on it, especially as regards the newer methods of driving, using a light hammer, striking very rapid blows; and also the bearing power of concrete piles.

Much has been said, some of it with good reason, on the subject of unfair requirements in the general clauses of contracts. This brings up some very large questions which should be considered by a committee representing the contractor, the engineer, the owner and the attorney. The committee would request that any members of the Institute, or others, wishing to be heard on that or kindred subjects, would publish their views and send copies to the committee, or send their views direct to the com-

* The results of these investigations will be published in the bulletins of the Engineering Experiment Station, Pennsylvania State College, State College, Pa.

mittee, which, it is hoped, may eventually be enlarged so as to give this subject adequate consideration.

It is becoming generally recognized among business men that it is true economy to treat all parties with fairness and justice.

Respectfully submitted,

Willis Whited, *Chairman.*
Lemuel Holmes.
A. N. Johnson.
A. M. Lovis.
Henry H. Quimby.

REVISION OF THE PROGRESS REPORT OF COMMITTEE ON REINFORCED CONCRETE HIGHWAY BRIDGES AND CULVERTS.

SUBMITTED AT THE TENTH CONVENTION.*

It has seemed best to the Committee to cover certain points in specifications for Reinforced Concrete Highway Bridges and Culverts by definite specifications, but certain other points must be varied according to the special circumstances of the case and, therefore, what is considered to be the best engineering practice is stated and recommended for adoption by the public. If any of the other national associations, of recognized standing, take up the same work, it would seem well to co-operate with them. It has been the policy of the Committee to adopt the standard specifications or recommended practice of any national society whose authority was generally admitted.

The following specifications and recommendations are submitted to the Institute, not as final, nor even as voicing the unanimous opinion of the members of the Committee, but merely as preliminary suggestions, in the hope of bringing out the discussion of doubtful points and information regarding questions which the Committee could not answer. Any information or criticism will be thankfully received and carefully considered.

SPECIFICATIONS.

1. The specifications for *Reinforcement* and *Cement* should conform to those of the American Society for Testing Materials.

2. The specifications for *Aggregates* should conform to those of this Institute.

3. The *Unit Stresses* in reinforced concrete, and reinforcement, should conform to the recommended practice of the Joint Committee.

RECOMMENDED PRACTICE.

DEAD LOADS.

4. The following are given as average weights per cu. ft. of the materials mentioned, but if the weights of the materials to be actually used are definitely known to be different from those given, the correct weights should be used.

* See JOURNAL, Vol. II, No. 2.

Materials.	Lbs. per Cu. Ft.
Earth filling	130
Plain concrete	150
Reinforced concrete	155
Steel	490
Cast iron	450
Vitrified brick	140
Common brick	125
Granite and limestone masonry	165
Sandstone	110
Macadam-Telford	150
Pine, fir, etc	42
Oak and yellow pine	48
Creosoted	60

5. The weight imposed by the earth filling, should be assumed to be equal to the weight of all the filling vertically above the portion of the structure in question, provided the depth of the fill does not exceed one-half the total length of the bridge, from face to face of the abutments, in which case, the depth of the fill should be assumed to be equal to one-half the length of the bridge, face to face of abutments. In the case of spandrel filled arches, if two planes inclined at an angle of 30° with the vertical one tangent to the arch extradus nearest each abutment, shall intersect below the surface of the road, only that part of the filling below said plane should be considered, otherwise the whole of the filling vertically above the arch should be assumed as resting upon it.*

This whole subject requires further investigation.

LIVE LOADS.

Class "A" Bridges—Main thoroughfares leading from large towns.

6. In view of the extensive introduction of the *heavy motor trucks and traction engines*, and the probable general use of such vehicles in the future, it is recommended that bridges on main thoroughfares and other roads which are likely to be used for heavy hauling, be designed to carry 25-ton trucks, with axles about 10 ft. c. to c., 17 tons on rear axle and 8 tons on fore axle; wheels about 5 ft. c. to c. Outside of the large cities it is recommended

* This paragraph is not approved by Mr. Lemuel Holmes nor by Mr. Henry H. Quimby.

that such vehicles be assumed to be on the bridge at any one time, the likelihood of more than two being on the bridge, in a position to produce maximum stresses at the same time, is so remote that this assumption is considered safe. It is advised that such very heavy loads be considered as occupying only the ordinary width of road, about 8 ft. in width and about 35 ft. in length. Congested traffic of heavily loaded wagons or motor trucks will rarely impose a load of more than 100 lb. per sq. ft., over a considerable area. The above mentioned 25-ton truck gives a load of about 180 lb. per sq. ft., on the area actually occupied, but it is considered extravagant to assume that a large bridge is covered with such heavy loads. 125 lb. per sq. ft. is thought ample to assume for the loading of spans more than 60 ft. long, in designing the trusses or main girders. It is thought to be safe to reduce this assumed load in the case of longer spans, to the following amounts:

Length of Span, ft.	Assumed Load, lb. per sq. ft.
80	110
100	100
125	90
150	85
200 and over	80

With all intermediate spans in proportion.

7. *Sidewalk Loads.*—The greatest load that is liable to be imposed on a bridge sidewalk, occurs when there is some excitement in the neighborhood, which attracts a large crowd, and for which the bridge affords an especially good point of view. In that case the crowd forms a compact mass, against the railing, not more than 4 ft. deep, making a load seldom exceeding 90 lb. per sq. ft., over a very considerable space. The remaining portion of the sidewalk may be covered by a moving crowd which can scarcely weigh more than 40 lb. per sq. ft. It may be advisable, sometimes, to so design sidewalk slabs, that if a street car or motor truck accidently gets upon the sidewalk, it will not go through. Such accidents are so rare, that it is thought safe to allow materials to be stressed nearly to the elastic limit in such cases.

8. *Class "B" Bridges.*—Although it is impossible to determine beforehand, especially in the newer parts of the country, whether any given road is to be used for heavy traffic, it seems extravagant,

at least in the cases of larger spans, to design bridges to carry much heavier loads than can be expected to come upon them. It is recommended that bridges of this class be designed to carry 17-ton trucks, with axles 10 ft. apart, 6 tons on the front and 11 tons on the rear axle. This will allow for a considerable overloading of existing motor trucks. It is further recommended, that only one truck be assumed to be on the bridge at one time, in designing the floor system, that it be assumed to cover a width of 8 ft. and a length of 35 ft. and that the remainder of the bridge is covered with a load of about 100 lb. per sq. ft., for spans up to 60 ft.

9. The longer spans, the trusses and main girders should be designed for the following loads:

Length of Span, ft.	Assumed Load, lb. per sq. ft.
80	90
100	80
125	75
150	65
200 and over	60

With intermediate spans in proportion.

10. *Sidewalks.*—Sidewalks should be designed to carry the same loads as in the case of Class "A" Bridges.

Special Bridges.

11. City bridges and bridges carrying traffic connected with mines, quarries, lumber regions, mills, manufactories, etc., require special consideration and should, of course, be designed to carry any load which can reasonably be expected to pass over them, bearing in mind the likelihood of heavy traction engines and motor trucks coming into extensive use in the not distant future.

Stringer Loading.

12. The maximum stress in a stringer, due to a wheel load, occurs evidently when the wheel is directly over it. It is not thought proper to assume any distribution of the load to adjacent stringers, unless the bottom reinforcement in the slab is made continuous. In that case the distribution is proportional to the relative stiffness of the slab and the stringers, said stiffnesses

being proportional to the moments of inertia, multiplied by the modulus of elasticity of materials and inversely proportional to the cube of the span. In determining this distribution, due account must be taken of the fact that deflection of the slab decreases toward the end of the stringers, and also of the fact, that whatever load is carried to the adjacent stringers, deflects them also. It is therefore recommended that wherever practicable the bottom reinforcement of slabs be made continuous over the stringers.

Slab Loading.

13. The distribution in a direction parallel to the supports of a concentrated load resting on a slab, supported at two opposite edges only, evidently depends upon the same principles as those mentioned under *Stringer Loading*. The main difference being that what corresponds to the stringer in the former case is of indefinite width in the present case. Adequate theoretical investigations of this question appear to be lacking. Some excellent experimental work has been done but not of sufficient extent to really prove much. Your Committee hopes to make a more complete report on this subject later. For the present it seems fair to assume that the distribution each side of a concentrated load is equal to about one-third the length of the span, and that the cross reinforcement should be designed accordingly, which would require it to have an area of at least one-half of the principal reinforcement. The distribution of a concentrated load through earth filling on the top of a slab, does not appear to be very well understood, the Committee has not, thus fark been able to find any records of adequate theoretical or experimental investigation of the subject. It is hoped to obtain more data later.

Bridges Carrying Electric Cars.

14. Electric traction is still in its infancy and nobody is able to forecast its future development. It seems probable, however, that it will not be profitable to run cars weighing more than 50 tons each, at a speed that would be permitted on any public road. If very high speeds are desired, the traction company will doubtless be required to operate over its own right-of-way. It is recommended that bridges carrying either urban or inter-urban electric cars be designed to carry 50-ton cars on two trucks,

spaced 20 ft. c. to c., each truck having two axles spaced 7 ft. c. to c. The Committee sees no reason for changing the customary practice of assuming that an axle load is distributed over 3 ties.

Loading on Arches.

15. The deflection of an arch being much less than that of a beam of the same length, the method recommended for determining the lateral distribution of a concentrated load over arch sheeting must be modified. It seems doubtful if the distribution in each direction can be greater than the thickness of the arch sheeting. This question should be investigated.

Stresses in Arches.

16. As all arches that are not provided with hinges, act as elastic arches until cracks are formed, due to excessive tension at some point or points in the concrete, it is manifestly proper to calculate the stresses in them according to the elastic theory. There is of course, no objection to using short methods in calculation or empirical rules, provided they give substantially the same results as the correct method, which is quite laborious.

17. As concrete is a very poor conductor of heat, it is not thought necessary in calculating reinforced concrete arches, to assume so much variation in temperature as is usual in designing steel structures, although the outside layers of concrete are of nearly the same temperature as the surrounding air, and those layers are stressed more heavily than any of the others, it is thought that an extreme variation of about 90 deg. in the Northern States is sufficient to allow for, in any case, and that can be reduced if the arch ring is thick or if there is much earth filling in the spandrels.

BEARING POWER OF PILES.

18. The usual formula for the safe bearing power of wooden piles is:

$$B = \frac{2WH}{S+1}$$

in which, W = Weight of hammer in pounds,
H = Height of fall of hammer in feet,
S = Penetration in inches per blow, average of last few blows.

Recent extensive tests on the bearing power of piles indicate that if this formula is used it may be with the understanding that it gives a factor of safety of 2, as regards the load which may cause a settlement of $\frac{1}{4}$ inch.

19. If reinforced concrete piles are molded before being driven, the head of the pile is usually cushioned more or less, and the pile is generally much heavier in proportion to the weight of the hammer than is the case with wooden piles. It is thought that a reduction of about 25 per cent should be made in the above formula unless a very heavy hammer is used.

20. If concrete piles are molded in place, measures should be taken to prevent damage to them by the driving of neighboring piles or otherwise.

21. The Committee hopes to make some further investigations in the subject before submitting a final report.

BEARING POWER OF SOILS.

22. This subject is under an investigation by a committee from the American Society of Civil Engineers. It may be some years before their final report is submitted and even that will be subject to revision from time to time, as human knowledge is extended. In the meantime the Committee would submit the following preliminary table:—

Material.	Safe Bearing Power, tons per sq. ft.
Quicksands and wet soils	0.1 to 1.0
Dry earth, according to depth below surface	1 to 3
Moderately dry clay confined	2 to 4
Dry stiff clay	4 to 6
Sand confined	2 to 6
Sand compact and cemented	4 to 8
Gravel cemented	8 to 12
Rock	25 to 200

Foundations should be carried down below frost unless they are on rock and thoroughly drained. Soil that contains the roots of plants is generally compressible. Undisturbed soil is much less compressible than filling or similar soil, even though it has been in place many years. The bearing power of sand, gravel and dry clay increases rapidly with the depth below the surface of the ground.

24. The pressure of earth against abutments, wingwalls and retaining walls varies so widely with the character and condition of the earth in question, that nothing more than a few general suggestions can be given. It is hoped that the Committee from the Society of Civil Engineers, which is studying the bearing power of soils, will take up this subject also. In ordinary cases where the filling is well drained its pressure will seldom be more than that of a liquid weighing 25 lb. per cu. ft., and is frequently much less. The pressure due to land slides is often several times this amount. If the filling is clayey and is allowed to become waterlogged, its lateral pressure is greatly increased. The effect of the freezing of the filling must also be considered.

REQUIRED WATERWAY.

25. The usual formula for waterway for culverts is:

$$a = C\sqrt[4]{A^3},$$

in which,

a = Required area of culvert in sq. ft.

A = Drainage area in acres,

C = A constant, depending on the length and character of the drainage area and may vary from about .3 to 2.0 in regions where the mean annual rainfall is about 50 in.

The capacity of the culvert will be much greater if the wingwalls are flush with the abutments and flare about 30 deg., the sides and bottom of the culvert are smooth and straight, and the bottom has a good slope.

If reliable information covering a number of years can be obtained regarding the adequacy of the old bridge or of another bridge crossing the same stream, it is much more useful in determining the size of a culvert or bridge than any formula.

Respectfully submitted by the Committee,

WILLIS WHITED, *Chairman.*
LEMUEL HOLMES.
A. N. JOHNSON.
A. M. LOVIS.
HENRY H. QUIMBY.*

* See Discussion, JOURNAL, Vol. II, No. 2.

DISCUSSION.

Loading on Public Bridges.

Mr. Whited.

Mr. Willis Whited.*—The engineer designing bridges for public use is always more or less at a loss to know what loading to assume for bridge floors; a few of the difficulties may be briefly summarized as follows:

Should he assume the heaviest known loads and design with the ordinary unit stresses? In large cities this is good practice, but the engineer is hardly justified even then in designing all the bridges in the city, large and small, to carry the heaviest monuments, castings, steel beams, building members, or stone columns that can be brought into the city, a limit must be fixed somewhere, beyond which the owner of the load is obliged to provide for its transportation by strengthening bridges, adopting different lines of travel, dividing up the load, using special trucks or otherwise. It seems a case of gross extravagance to add a large percentage to the cost of a bridge in order that it may carry a load heavier than is likely to be brought to it once in five years, especially as the necessity for carrying such loads is rarely absolute. It would seem that all bridges, except boulevards and bridges in parks, should be designed to carry street cars, whether they are actually used or not. It is impossible to accurately forecast the growth of a city or to know what traffic is to come on a bridge in the future, twenty-five years for example. The street car is with us and likely to stay for many years and must of course be provided for, and we can only guess as to the weight they are likely to attain, although it may be said that the bridge engineer can practically limit their weights by limiting the strength of the bridges, as the owners of the street railways will find it more economical to use cars no heavier than those now in use rather than rebuild all the bridges they have to cross, even though it would otherwise be advantageous to build heavier

* Bridge Engineer, Penna. State Highway Dept., Harrisburg, Pa.

cars. It would probably be desirable to build bridges in parks and along boulevards no heavier than is necessary to carry the heavy class of automobiles and the road rollers used for paving such roads, thus placing an obstacle in the way of the street railways corporations' grabbing such roads by "hook or by crook" to the detriment of the traffic they were intended to carry; the same may be said of highways between towns, which are only wide enough to carry two lines of traffic, these considerations would seem to make it advisable to build bridges in public streets to carry inter-urban cars weighing fifty tons each, this is much heavier than ordinary cars, but it must be borne in mind that the use of suburban cars is destined to a wide extension, and almost any street in a city is liable at some future time to be called upon to carry them. It must also be remembered that, owing to the fact that people will crowd to one side or one end of a car, the distribution of the load on the various wheels is often quite different from that provided for in the calculations. Mr. Whited.

Roads between large towns are, it seems to me, destined eventually to be subjected to the traffic of heavy motor trucks which will carry miscellaneous freight from the store, manufactory and warehouse in the city, to the residence of the consignee, be it in the country or in the city; the same trucks will carry farm produce, lumber and other materials and manufactured goods from the small town or the country to the warehouse in the city.

So long as rubber is used for tires, and the present materials for paving, it is hardly to be feared that the gross weight of the truck will greatly exceed twenty tons, with about two-thirds of the weight on the rear axle. I am also inclined to think that loads due to traction engines will not exceed that of the motor truck. In my opinion, all bridges on important thoroughfares should be designed to carry that load.

There are roads in remote districts which could not, without extravagance, be built to carry twenty-ton trucks, but the road roller and traction engine are a permanent institution and I would recommend a twelve-ton roller as the lightest load for which bridges on public roads should be designed to carry.

There is always difficulty in deciding whether a given district will always be remote or not. I would recommend resolving all cases of doubt in favor of the heavier loads. Cases often arise

Mr. Whited.

in which designing a very large bridge for very heavy loads would add so much to the cost as to be an unreasonable burden on the taxpayers, while there is another route not requiring such an expensive bridge which could carry exceptionally heavy loads without undue inconvenience to the public, such points are worth considering, but notices should be placed on such bridges warning vehicles carrying loads exceeding a certain amount to take some other route.

The driver of a heavy load may feel like taking a chance, nevertheless, there is plenty of law holding him responsible for accidents happening in such cases.

The question often arises in considering bridges on remote roads, whether it would not be better economy to build them to carry lighter loads. They could be strengthened at later dates by reconstructing the floors.

The loadings on the roadways of public bridges is, as is well known, liable to important changes in the not distant future, owing to the changes in the conditions of transportation, and for that reason it is well to provide for possible future changes in designing bridges. It is certain, however, that sidewalk loading cannot change as the years pass, either in character or amount.

The greatest load imposed on a sidewalk is, of course, a dense crowd, but although loads of 150 lb. per square foot have been made by special exertion, such loadings are so extremely rare that it seems to me not unreasonable to allow stresses under such loading to approach the elastic limit. An ordinary standing crowd will rarely weigh more than 80 lb. per square foot and the only occasion on which crowds even of that density will be gathered will be in cases of some local excitement, such as accidents or similar occurrence in the water or the roadways, in such cases the densest part of a standing crowd can hardly be more than four deep, those in the rear being unable to see anything will keep moving, and a crowd with freedom enough to move at all, even though composed exclusively of men, will rarely exceed 40 pounds per square foot. I am, therefore, of the opinion that an assumed sidewalk load of 100 pounds per square foot, four feet deep, either next to the curb or next to the railing, with the rest of the sidewalk covered with a load of 50 pounds per square foot, is ample for any kind of a bridge, but I think it should be provided for in all

bridges, whether urban or not, because any district sufficiently populous to require sidewalks at all on the bridge will occasionally produce crowds of considerable density. Dense crowds of much greater depth occasionally gather to listen to musicians or street speakers, but such things are never allowed on bridges. Mr. Whited.

With regard to the vibration due to foot traffic, that cannot happen unless a very large crowd keeps in accurate step and it synchronizes with the rate of vibration of the bridge or some important members of it. It is customary for military bodies to break step when crossing bridges, and I think that the additional stress due to such vibration can be safely ignored. Impact due to the surging of crowds can, I think, be left out of consideration, for the reason that the effect is very small unless the whole crowd moves as one man, which is scarcely possible.

Regarding the weight of snow, which sometimes accumulates on sidewalks, I would say that it is hardly possible for crowds to become nearly so dense in cold weather, on account of the necessity for everybody to keep moving, as the weight of ice or snow seldom exceeds 13 pounds per square foot I think it is hardly necessary to add anything for its weight. If the bridge is at the foot of a steep grade it may happen that street cars, escaping from the control of the motorman, will leave the rails at the foot of the hill and go over on the sidewalk; this accident occasionally happens even on a level track, as such accidents are apt to be accompanied by heavy loss of life, especially in large cities, it may be worth while to allow for such accidents in designing sidewalks, as it is impracticable to build railings of sufficient strength to resist the impact of street cars or heavy automobiles running at a high rate of speed. The best way to guard against the above-mentioned accidents is to make the curb sufficiently high and strong enough to prevent such vehicles from getting on the sidewalks, for this purpose the curbs should never be less than 6 inches high, and may be more, since a street car mounting the curb is almost certain to do much damage, requiring extensive construction of the sidewalks. I think it sufficient in providing for such occurrences to assume a unit stress, nearly, if not quite, three times the normal, this might result in badly damaged sidewalks but would prevent loss of life, and would not involve extravagance in the cost of the bridge.

Mr. Whited.

The planks or other details of the sidewalk should be designed to carry not less than 250 pounds, because persons of that weight are not uncommon, and a person in walking always places his whole weight on one foot. Plank walks are rapidly going out of use, but they will be with us for many years to come, and in fixing the thickness of a plank a reasonable allowance must be made for wear and deterioration.

The automobile and auto truck according to present indications are destined to furnish a large part of highway traffic. It is the duty of public officials to protect and care for all reasonable and legitimate traffic on public roads. It is therefore well to consider the accidents to which different kinds of vehicles are liable.

Horse-drawn vehicles, even in the case of runaways, must follow the horses. Horses, except when under the spell of frenzied fear, will never bolt over a fence, it would therefore seem that where a curb is surmounted by a fence, *i. e.*, where there is no sidewalk, all that is required is a reasonably high fence and a curb of about 6 inches height. If, however, there is a sidewalk it is well to make the curb about 16 inches high, then in rare cases in which horses mount the sidewalks, the sidewalk railings, if of reasonably substantial construction, will prevent serious accidents, but the sidewalks should be made sufficiently strong to carry the weight of a horse without greatly exceeding the elastic limit.

As horses drawing very heavy loads rarely run away except they break loose from the loads, it is hardly necessary to design the sidewalks for carrying anything more than the ordinary wagon.

In the case of automobiles, however, the conditions are different, the ordinary touring car is very often driven at a high rate of speed, occasionally the driver loses his head or the stearing gear gets out of order, in which case the car is liable to mount the sidewalk. The best preventive of this accident would perhaps be to make the curb about 16 inches high and sufficiently substantial to resist the impact of the car, or rather so that the car will be wrecked rather than breaking it down or mounting it.

There are objections to this, among the principal being their unsightly appearance, and the difficulty of passing from one side of the street to the other. It is hardly practicable to build a

fence or even a reinforced-concrete parapet wall sufficiently strong to withstand the impact from a high-powered automobile. Mr. Whited.

In the majority of such cases the automobile strikes the curb at such an angle that a 10-inch curb is sufficiently high to prevent accident, but there may be cases in which there is a sharp curve on the approach, or the bridge is very wide, that will make the car more liable to strike very nearly square, in which case a higher curb is advisable.

The auto truck is much heavier than the pleasure vehicle but travels at a much lower rate of speed, it is much more likely therefore that the driver will be aware of accidents to any part of his machine in time to stop before the truck mounts the curb.

As to the railings I think that railings should be made to resist a lateral pressure applied at their tops equal to one-fourth the weight of a row of men leaning against it, or approximately 40 pounds per lineal foot. If the railing is not so constructed that it is impossible for men to climb upon it, they should be strong enough to sustain a vertical load of at least 100 pounds per lineal foot.

If the railings are on a sidewalk there should be no opening more than 6 inches wide which a child could get through, either by walking between the balustrades or falling down and rolling underneath.

In the case of a bridge without sidewalks or bridges so situated that a child falling through them would suffer no serious injury, some relaxation of this rule might be allowed.

DISCUSSION.

Mr. Hatt. PROFESSOR W. K. HATT.—Some concrete bridges in Indiana are designed to carry a 100 ton electric locomotive; and interurban bridges are being designed for a string of 100 ton locomotives. We are designing some highway bridges for the same load.

I would ask whether the unit stresses are recommended in accordance with the Joint Committee report for concrete and reinforced concrete? Is an arch ring 4 ft. thick, reinforced on one side only with $\frac{3}{4}$ in. bars spaced 12 in., plain concrete or reinforced concrete?

Mr. Mensch. MR. L. J. MENSCH.—It is reinforced concrete absolutely.

Mr. Hatt. PROFESSOR HATT.—In the event that the pressure on that arch ring is slightly eccentric and you have a very slight gradation of stress from one side to the other, would you use the stress for plain concrete, say 350 lbs., or stresses for reinforced concrete of 700 lbs. per sq. in.?

Mr. Mensch. MR. MENSCH.—If you put the concrete arch in one day you cannot figure on any stress in the concrete in tension.

Mr. Hatt. PROFESSOR HATT.—I am speaking now of the compressive stress in the concrete.

Mr. Mensch. MR. MENSCH.—I cannot see by the insertion of steel rods in concrete that you raise the strength of the concrete. You might raise the strength of the combination but you cannot raise the strength of the concrete simply by imbedding steel rods.

Mr. Hatt. PROFESSOR HATT.—I am not speaking of that, Mr. Mensch. You misunderstand me. It is a question of the unit stress to be allowed in designing arches, and I ask you or I ask the committee if they will recommend a stress of 750 lbs. per sq. in. compression on that concrete arch ring under conditions stated, or whether they would allow only a unit stress for plain concrete.

Mr. Mensch. MR. MENSCH.—I wish to answer Professor Hatt, that in Europe the railroad bridges are built of limestone or masonry without any reinforcement allowed. The Government allows a stress of 750 lbs. on compression. Tensile stresses are taken care of in those 750 lbs.

MR. WHITED.—The committee will certainly act upon the suggestion of Professor Hatt and if anybody has any further suggestions in that line or in any other line, we would be delighted to hear from them. So far there has been scarcely any discussion of these specifications. Answering Professor Hatt, it seems to me that the reinforcement is so small it should not be considered. Mr. Whited.

TESTS OF CIRCULAR AND EGG-SHAPED REINFORCED CONCRETE SEWER PIPE.

By A. T. Goldbeck.*

The use of reinforced concrete pipe is an innovation in sewer construction in the City of Philadelphia, and its introduction led to the present investigation. When a new material or a new construction is used, prejudice and doubt regarding its suitability are likely to exist, and queries relative to particular phases of the construction arise almost invariably. There is no better way to settle these doubts and answer these queries than by actual demonstration in the form of suitable tests.

A number of questions presented themselves concerning the reinforced concrete pipe first used in Philadelphia. With the type of pipe used, what was the minimum time permissible for curing? What was the most desirable method of curing? Was the specified system of concrete cradle foundation construction, justified, or necessary? Were the pipes sufficiently impervious and sufficiently strong to resist reasonable internal pressure? These and other questions required investigation.

Description of Pipes.

Two kinds of pipes were made; one was circular in cross-section, 36 in. in diameter and 4 in. thick, and the other was egg-shaped, having a long diameter of 36 in., a short diameter of 24 in., and was $3\frac{1}{2}$ in. thick. The egg-shaped section was provided with a flat base, 8 in. wide.

The materials used in the manufacture were a well known brand of Portland cement from the Lehigh Valley district which satisfactorily passed the standard tests, New Jersey bank sand and well graded $\frac{3}{4}$ in. crusher run trap. In a few of the pipes, crushed shale of undesirable qualities was used, but as it was not suitable, its use was discontinued. Four experimental sections also were made of concrete containing $\frac{3}{4}$ in. graded, air-

* Assistant Engineer, Testing Laboratory, Bureau of Surveys, Philadelphia.

cooled blast furnace slag having a weight of 74.7 lbs. per cu. ft., when shaken to refusal in a cu. ft. measure. The concrete was machine mixed in the approximate proportions, 1:2:3½.

The pipes were reinforced with triangular mesh placed 1¼ in. from the inner surface and described in the following table:

DESCRIPTION OF REINFORCING.

	Used in 24 x 36 in. egg shape.	Used in 36 in. circular.
No. of strands, longitudinal	1	2
Gage of longitudinal strands	6	8
Gage of cross wires	12½	12½
Sectional area, long. sq. in	0.087	0.124
Sectional area, cross wires sq. in	0.038	0.038
Cross sect. area per ft. width	0.110	0.146
Wt. per 100 sq. ft	50 lbs.	63 lbs.
Tensile strength	80,000 to 90,000 lbs. per sq. in.	

After pouring in special steel molds, the pipes were allowed to harden, some of them under atmospheric conditions, others when subjected to a hot vapor bath continued for a period of about 12 hours. The application of hot vapor at a temperature of about 100 deg. F. accelerates the hardening and permits the removal of the forms on the day after pouring without danger of injury to the pipe during the removal. It thus permits the manufacturer to economize on molds. It likewise enables him to make pipes in freezing weather. After the molds were stripped the green concrete was kept damp by daily sprinkling for one week.

EXTERNAL LOADING TESTS.

Method of Bedding.—Two methods of bedding the pipes were employed: (*a*) damp sand bedding and (*b*) bedding in a concrete cradle. Three in. of sand or concrete was used under the pipes extending on each side to a height of ¼ of the vertical diameter. Where a sand bedding was used, the pipe was placed on the layer of lightly tamped sand and sand was heaped and tamped around the sides to the proper height. In the case of the concrete cradle bedding, the pipe was placed in the soft concrete and it remained there until tested. In some cases the cradle was only one day old when the pipe was tested, but generally was older than this.

The specifications under which the pipes were laid, called for a concrete cradle such as described, but the sand bedding tests were made in order to inquire into the necessity for the concrete cradle construction.

Method of Loading.—The load was transmitted to the specimen under test through a cushion of dampened sand, contained in a bottomless box resting on the specimen. The box was braced properly with tie-rods to prevent its spreading under load. A wooden plunger, carrying a timber platform rested on the sand, and the load was obtained by piling bags of cement on the platform. This method was adopted in the absence of a testing machine of the desired type and capacity. It was a very expensive method but was quite effective. The loads were applied at the rate of 10 bags (950 lbs.) in about 5 minutes and vertical distortion readings were taken after each 10 bag increment. Figs. 1 and 2 and the photographs show the method of loading in detail.

Method of Measuring Vertical Distortion.—In order to note the elastic behavior of the pipes under load, distortions of the vertical diameter and in some cases distortions of the horizontal diameter were measured. This was done in two ways (1) by the use of an Ames dial reading to 0.001 in. and (2) by the measurement of the middle ordinate between 2 light wooden battens sprung into place between the crown and invert. A very slight movement in vertical distortion produces a comparatively large increase in the middle ordinate. This device is extremely cheap and convenient to attach. It is held in place entirely by the compression in the battens. The middle ordinate readings were taken on a scale reading to 0.01 in. and were readily converted to vertical deflections by calibration. This device was used merely as an expedient during the temporary lack of an Ames dial. It, however, is quite accurate and is capable of refined application in other fields of testing where deflection measurements must be made.

THE TESTS.

It was the aim to test a single pipe of each kind at the age of 7, 14, 21 and 28 days, bedded in sand and concrete and cured with and without steam, and this program was followed as com-

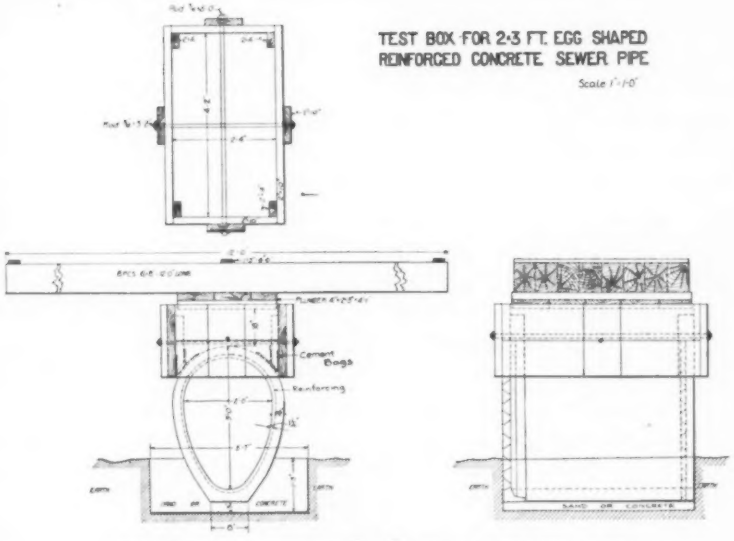


FIG. 1—LOAD TEST ON 2 X 3 FT. EGG-SHAPED PIPE.

pletely as weather and other conditions would permit. The pipes were placed in their cradle at least one day before testing. Vertical distortion readings were not taken until the sand box and loading platform were mounted into place, and the initial readings were made under the weight of sand box and platform loading.

After each increment of 950 lbs., distortions were read, and the surface carefully searched for hair cracks.

CURVES OF VERTICAL DISTORTION.

The readings for vertical distortion are plotted as abscissas on Figs. 3, 4, 5 and 6, and the ordinates are the corresponding leads carried on the test section of pipe, 4 ft. long. The curves thus obtained indicate several facts concerning the pipes.

Considering first the unsteamed pipes, it will be seen that stiffness is gained with age, and the load under which the first hair cracks became visible likewise advanced with the age of the pipe. This is to be expected, and is in agreement with other tests on reinforced concrete. Note the great increase in stiffness resulting from the use of a concrete cradle rather than sand bedding. As an accompanying result, cracking occurs at a much higher load when a concrete cradle is used than where the pipe is merely bedded in sand. This is true of egg-shaped and of circular sections, but is particularly true of the circular pipes. It was required by the city specifications that the pipes be able to withstand an external load of 1000 lbs. per sq. ft. without showing cracks. This was a reasonable requirement for the conditions under which the pipes were laid. It will be noted that the circular sections embedded in sand, when unsteamed, are unable to pass the specifications at 6 and 15 days and lie uncomfortably close to the requirements at 21 days. When a concrete cradle is used, however, no difficulty whatever is encountered in exceeding the loading requirements. In constructing a sewer of circular pipes such as these, with a likelihood of their being subjected to a load of 1000 lbs. per sq. ft., a concrete cradle, extending in height $\frac{1}{4}$ of the diameter of the pipe above the base is recommended as necessary in order to avoid cracks.

As might be expected, the egg-shaped pipes were stiffer than the circular pipes, and were afforded not as much extra

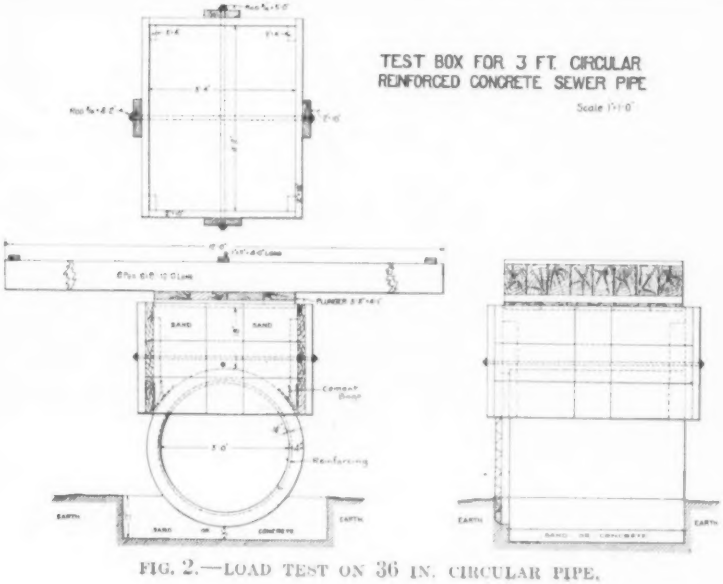


FIG. 2.—LOAD TEST ON 36 IN. CIRCULAR PIPE.

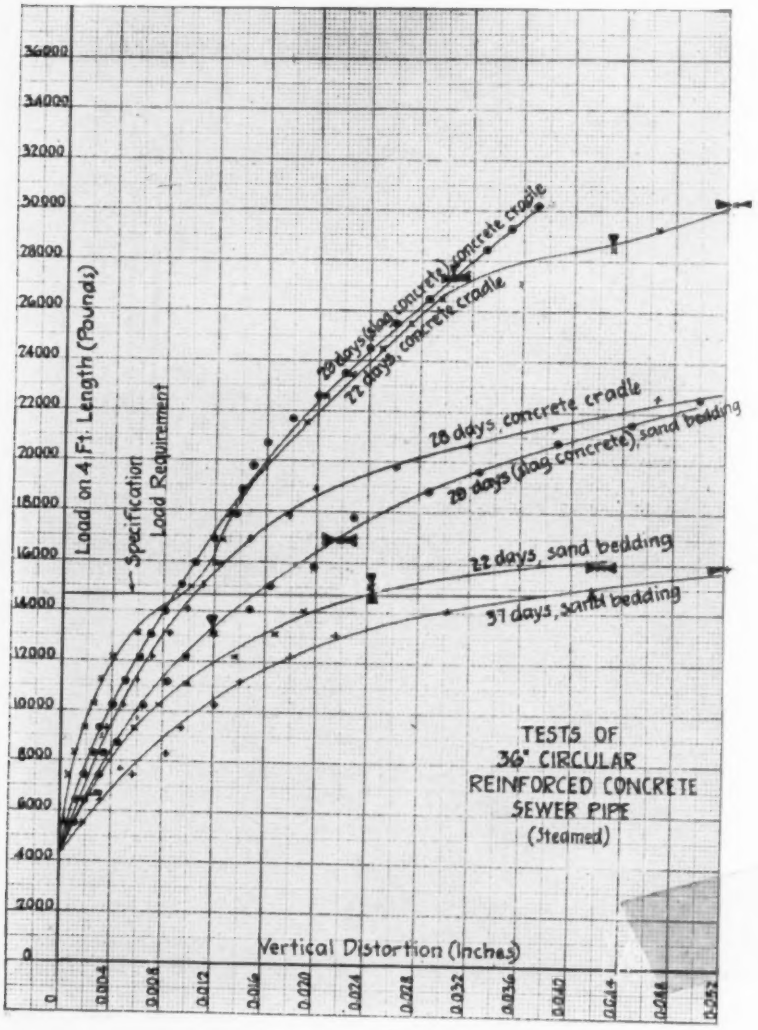

Load on 4 Ft. Length (Pounds)
Specification Load Requirement
28 days (slag concrete) concrete cradle
22 days, concrete cradle
28 days, concrete cradle
28 days (slag concrete), sand bedding
22 days, sand bedding
37 days, sand bedding
TESTS OF 36" CIRCULAR REINFORCED CONCRETE SEWER PIPE (Steamed)
Vertical Distortion (Inches)

FIG. 3.

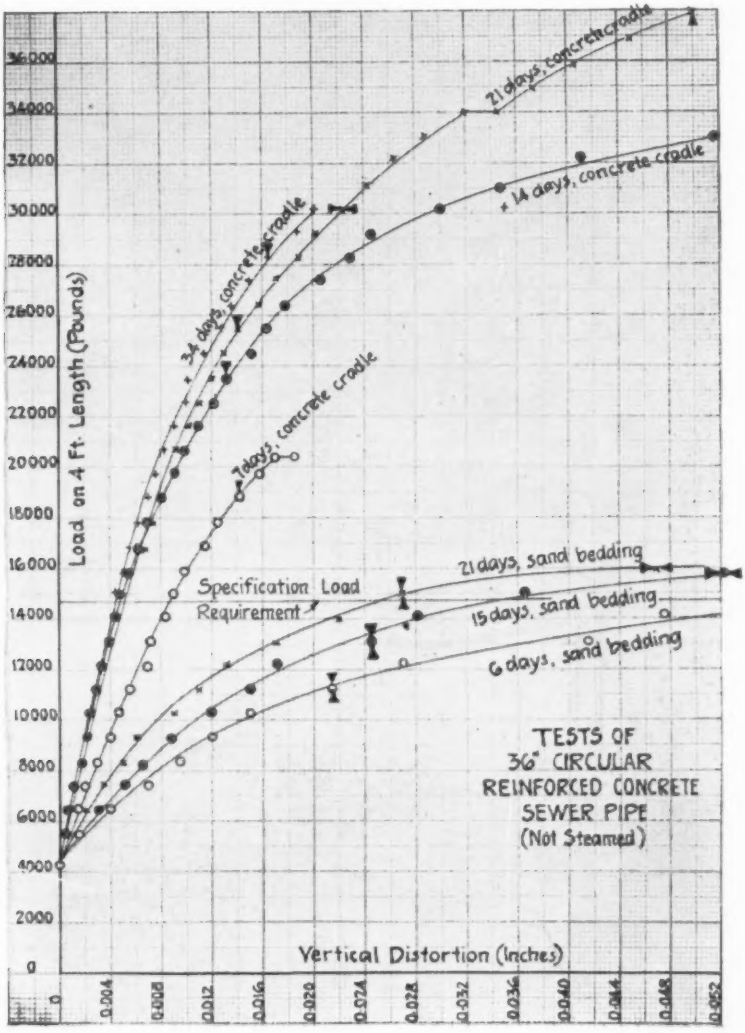

Load on 4 Ft. Length (Pounds)
36000
34000
32000
30000
28000
26000
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
21 days, concrete cradle
14 days, concrete cradle
34 days, concrete cradle
7 days, concrete cradle
Specification Load Requirement
21 days, sand bedding
15 days, sand bedding
6 days, sand bedding
TESTS OF 36" CIRCULAR REINFORCED CONCRETE SEWER PIPE (Not Steamed)
Vertical Distortion (Inches)
0
0.004
0.008
0.012
0.016
0.020
0.024
0.028
0.032
0.036
0.040
0.044
0.048
0.052

FIG. 4.

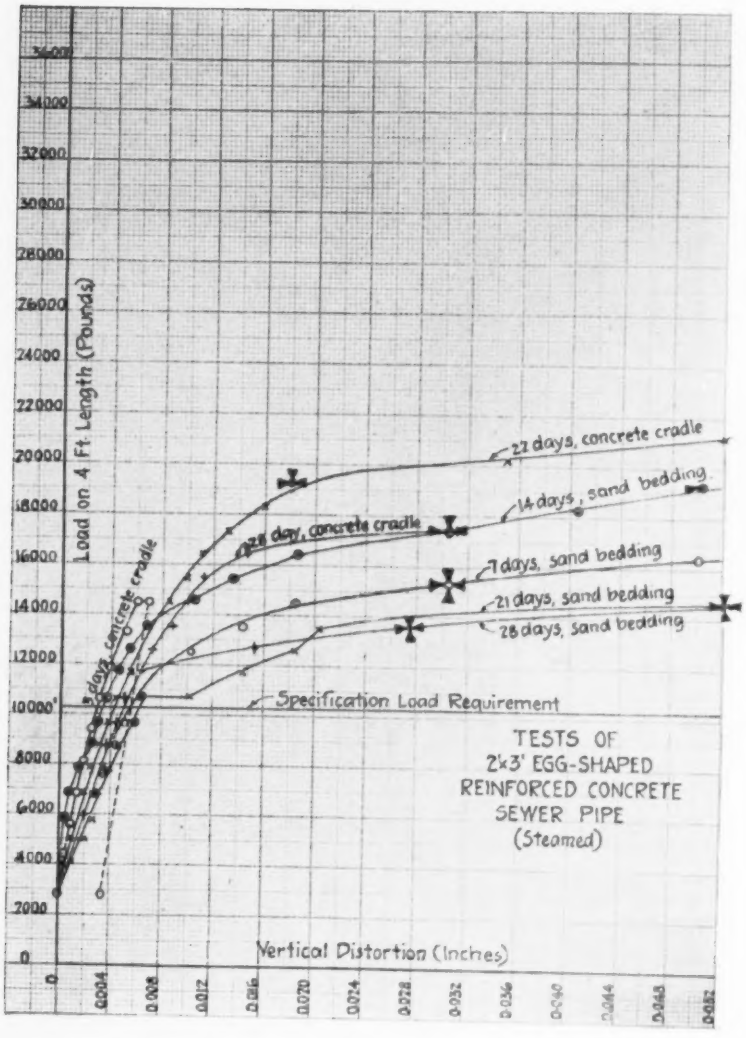

Load on 4 Ft. Length (Pounds)
34000
32000
30000
28000
26000
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
Vertical Distortion (Inches)
0.004
0.008
0.012
0.016
0.020
0.024
0.028
0.032
0.036
0.040
0.044
0.048
0.052
22 days, concrete cradle
14 days, sand bedding
28 day, concrete cradle
7 days, sand bedding
21 days, sand bedding
28 days, sand bedding
7 days, concrete cradle
Specification Load Requirement
TESTS OF
2'x3' EGG-SHAPED
REINFORCED CONCRETE
SEWER PIPE
(Steamed)

FIG. 5.

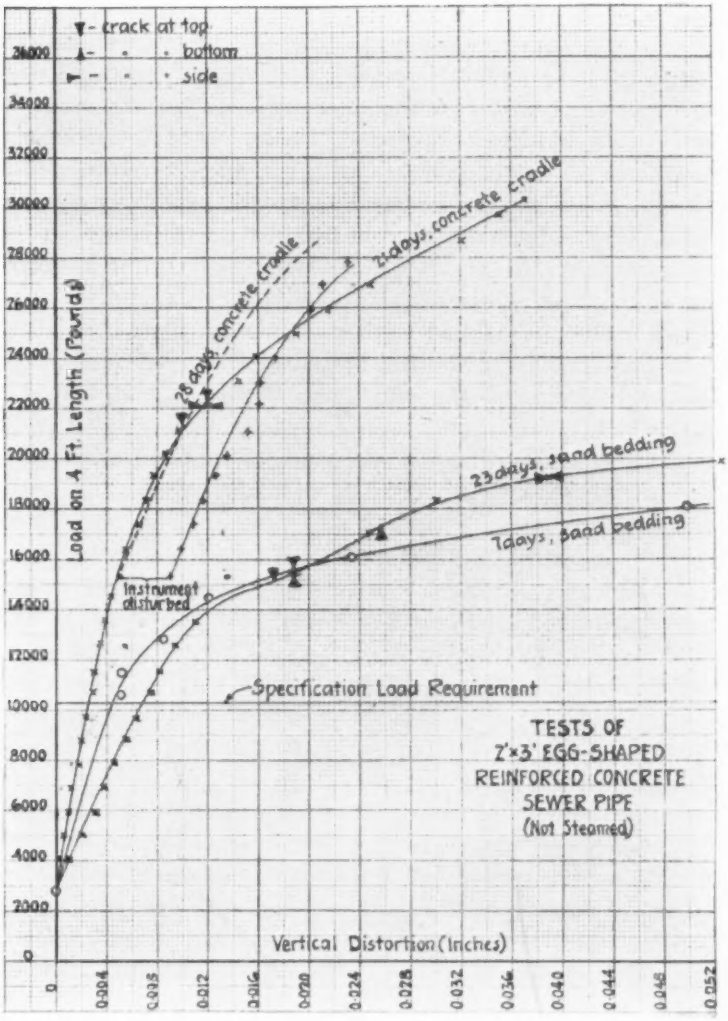

▼- crack at top
▲- " " bottom
T - " " side
Load on 4 Ft. Length (Pounds)
Vertical Distortion (Inches)
28 days, concrete cradle
21 days, concrete cradle
23 days, sand bedding
7 days, sand bedding
Instrument disturbed
Specification Load Requirement
TESTS OF
2'×3' EGG-SHAPED
REINFORCED CONCRETE
SEWER PIPE
(Not Steamed)

FIG. 6.

support by the concrete base as were circular pipes. However, even in the egg-shaped sections, the extra stiffness and resistance against cracking provided by the concrete warrant its use.

A comparison of the steamed and unsteamed pipes is of interest. There seems to be an indication that the steam cured pipes, that have had their setting accelerated, are more uncertain in their strength at the later periods than are the pipes cured under normal conditions. The strength of the normally cured pipes follows a fairly definite law of increase in stiffness with age. The steam cured pipes on the other hand are extremely erratic, some of the older pipes exhibiting less stiffness than those of lesser age. The steam method of initial curing is open to some question, and if allowed, should be controlled very carefully in order not to drive out the moisture from the concrete before the cement has attained its set.

The behavior of the experimental pipes made with blast furnace slag coarse aggregate is interesting in indicating that the strength and stiffness of the pipes of this material equalled or even slightly exceed those of pipes made with crushed trap.

HAIR CRACKS.

It is highly undesirable to subject a reinforced concrete sewer pipe to a load such that hair cracks are likely to form, because of the danger of corrosion of the reinforcing metal that must result. That which causes a visible hair to form is to be regarded therefore as the ultimate strength of the pipe, even though the load necessary to cause actual collapse may be much higher.

On the curves, the loads at which cracks were first noticed are indicated by arrow heads, and the position of the arrow indicates the position of the cracks, whether at the bottom, top or sides of the pipes. The pipes embedded in sand almost invariably failed by cracking at the bottom, inner surface or invert, and in the circular pipes a crack was generally first noticed at the top inner face. The side cracks which occurred on the outer surface of the sand embedded pipes were sometimes noticed simultaneously with the top and bottom cracks, and sometimes not until after the top and bottom cracks had formed.

When a concrete cradle is used, the first crack almost inva-

riably occurs at the top, and is accompanied sometimes by side cracking on the outside. Cracks do not form at the extreme bottom when the pipe is bedded in concrete, although under very high loads, and after all other cracks have formed at the top and sides, inside cracks will develop just above the level of the concrete cradle.

A very green concrete cradle, even when not more than one day old, adds greatly to the strength of the pipe.

The order of formation and position of the cracks are very comforting, particularly when a concrete cradle is used. Thus it has been seen that the invert remains intact even long after the pipe has been loaded sufficiently to crack it at the crown. The sand bedded pipes, however, cracked at the invert under quite a low load. This is an additional argument for the use of a concrete cradle rather than earth bedding. The fact that cracking occurs at the top or bottom on the inner surface before it occurs at the sides on the outer surface is significant. It shows that the steel is placed correctly to provide for the tension at the dangerous sections of the pipe. Increased resistance against complete collapse would be obtained perhaps by shaping the steel to approach the outer surface at the sides, but collapsing strength is not of much interest as it is well above the load required to produce the first visible crack. The extra expense which would be involved in this procedure is therefore not warranted.

TESTS FOR IMPERMEABILITY AND RESISTANCE AGAINST INTERNAL PRESSURE.

In a sewer of the size tested, a leakage amounting to more than about 10,000 gallons per mile in 24 hours, might be regarded well as the maximum allowable limit. A reasonably tight sewer placed in the wettest soil at a considerable depth should not admit more than this amount of ground water. The sewer not only should be safe against excessive inward leakage but it should likewise be capable of resisting a possible interior pressure due to flooding without cracking and thereby developing large leaks.

The specifications required, therefore, that excessive leakage should not occur under a pressure of 5 lbs. per sq. in. and

should not crack nor burst under a pressure of 10 lbs. per sq. in. interior pressure.

In order to make the hydraulic test, two sections of pipe were grouted together in the same manner in which they would be treated in the trench. Heads made of steel plates were bolted

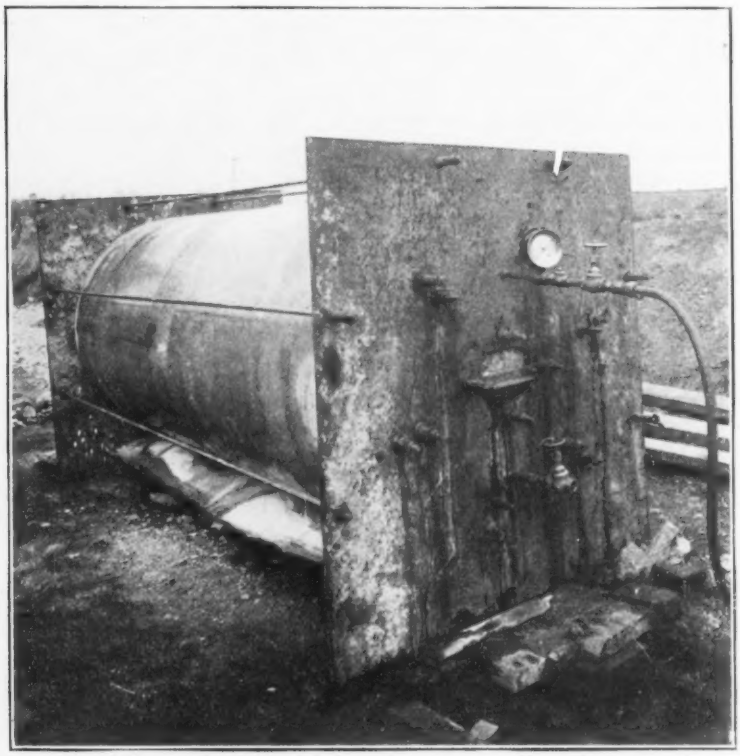


FIG. 7.—HYDRAULIC TEST ON 36 IN. CIRCULAR REINFORCED CONCRETE PIPE.

to the ends by means of long bolts extending through the pipe and along the sides. Rubber carriage tires of the correct diameter were used as gaskets to prevent leakage at the ends. The leakage was collected in a tin drip pan and measured. The grouted joint connecting the two sections was about 2 weeks old when the first test was made, and the pipes were 30 days old. See Fig. 7.

The internal pressure required to crack the pipes was 25 lbs. per sq. in.

A corresponding test made on a slag concrete pipe showed a leakage of 5470 gal. per mile per 24 hrs., and required an internal pressure of 32 lbs. per sq. in. to cause rupture. Most of this leakage occurred at the grouted joint.

The above results for the stone pipe are not particularly good. Concrete as rich as that used in the manufacture of these pipes can be made practically water-tight against 5 lbs. per sq. in. pressure. Most of the leakage occurred in the upper half of the pipes as molded, showing that a difference in density existed in the concrete, depending on its position in the forms. However, in the trench, leaks such as these due to somewhat porous concrete and not to actual cracks in the concrete would soon become filled with silt and no undue leakage should thereafter result.

RESULTS OF PERMEABILITY TEST.

Age, days.	Pressure, lbs.	Duration of Test, hours.	Leakage, gal. per mile per 24 hrs.
30	5	1	7,920
31	5	1½	13,860
31	5	1	11,900
32	5	1	15,840
33	5	1	10,890
34	10	½	27,700
40	10	1*	13,860

* Muddy water.

CONCLUSIONS.

The following conclusions seem warranted:

1. Reinforced concrete pipes, when properly made and properly cured, are very satisfactory for the construction of sewers.

2. The use of a concrete cradle increases the stiffness of the pipe greatly and raises the load under which initial cracking takes place.

3. For sections of the size tested and for smaller sections, the placing of the reinforcing near the inner face throughout the entire circumference seems to be justified.

4. The matter of curing should be controlled very carefully, as much stronger pipes will result if care is taken to keep them thoroughly wet, preferably for a period of two weeks.

The manufacture of concrete pipe is a process that can be controlled very precisely, and the character of the concrete produced should be much superior to that made in the average con-

RESULTS OF TESTS ON REINFORCED CONCRETE SEWER PIPE 4 FT. 0 IN. LONG.

PIPES STEAMED AT ABOUT 100° F. FOR ABOUT 12 HOURS BEFORE REMOVAL FROM FORMS. 24 X 36 IN. EGG-SHAPED PIPES.

How Supported.	Age, days.	Load, in lbs., at First Crack.	Vertical Distortion at First Crack.	Total Load, in lbs., Applied. No Collapse.	Vertical Distribution.	Position of First Crack.
Concrete....	3	.....	.....	14,500	0.008	No cracks observed.
Cradle....	14	21,150	0.029	25,950	0.08	Top.
Cradle....	22	19,250	0.019	24,950	0.130	Top and sides.
Cradle....	28	16,400	0.015	17,350	0.031	Top.
Sand.....	7	15,350	0.031	16,400	0.051	Top, bottom, sides.
Bedding...	14	17,350	0.034	19,250	0.05	Top, bottom, right side.
Bedding...	21	14,500	0.06	19,250	0.214	Top, bottom, sides.
Bedding...	28	13,550	0.028	19,250	0.262	Top, bottom, sides.
		36 IN. CIRCULAR PIPES.				
Concrete:						
Cradle....	22	28,300	0.043	30,250	0.053	Top.
Cradle....	28	24,500	0.064	29,250	0.106	Top and sides.
Cradle....	29*	27,350	0.032	30,200	0.037	Top and sides.
Sand:						
Bedding...	22	15,000	0.026	15,950	0.044	Top and bottom.
Bedding...	29*	13,100	0.012	23,550	0.054	Top.
Bedding...	37	15,050	0.04	20,750	0.11	Bottom.
		PIPES NOT STEAMED WHILE IN FORMS. 24 X 36 IN. EGG-SHAPED PIPE.				
Concrete:						
Cradle....	21	22,150	0.014	30,300	0.038	Side and top.
Cradle....	28	21,150	0.011	28,750	0.022	Top.
Sand:						
Bedding...	7	15,300	0.02	21,000	0.08	Bottom.
Bedding...	23	15,450	0.02	21,150	0.106	Top.
		36 IN. CIRCULAR PIPES.				
Concrete....	7	18,800	0.017	20,300	0.02	Top.
Cradle....	14	23,550	0.015	33,050	0.05	Top.
Cradle....	21	25,450	0.016	42,550	0.08	Top.
Cradle....	34	28,300	0.016	30,200	0.03	Top.
Sand.....	6	11,200	0.028	33,000	Est. $\frac{1}{4}$ in.	Top and bottom.
Bedding...	15	13,100	0.028	16,900	0.067	Top and bottom.
Bedding...	21	15,000	0.03	19,750	Abt. 0.16	Top and bottom.

* Slag concrete pipe.

crete construction. The quality of the various materials available should be investigated very carefully before use. Much can be done toward producing stronger concrete by the selection of first class sand and coarse aggregate. But the manufacturer

should not stop at merely obtaining satisfactory constituent materials, he should go further. He should find out by very simple density tests how he should combine these constituent materials in order to obtain with them concrete of the greatest density and greatest strength, for it must be emphasized that dense concrete is vitally necessary for concrete sewer construction.

TESTS ON SAND USED IN CONCRETE SEWER PIPE.

Sp. Gravity..... 2.654
Per cent voids (loose).....42.5
Wt. per cu. ft. (loose).....95.7

Strength Tests—Tension..... 1:3 Mortar

	7 Days	28 Days	3 Mos.
Bank sand.....	199	344	415
Ottawa sand...	330	460	493
Ratio sand.....	60.3 per cent	72.7	84.3

Compressive Strength 1 : 3 Mortar

	7 Days	28 Days
Bank sand.....	850	1842
Ottawa sand...1194		2625
Ratio.....	71.1	70.2

	Per Cent.
Passing ¼	99.3
" ⅛	96.2
" 10	93.9
" 20	82.7
" 30	72.0
" 40	62.8
" 50	35.5
" 80	9.6
" 100	4.1
" 200	0.5
Silt	0.0

CRUSHING STRENGTH TESTS ON CONCRETE.

Cylinders made from Concrete used in Pipe.

No.	Age.	Crushing Strength.	
1	82	2260	Steamed 10 hrs. buried in ground 77 days.
2	82	2910	" 10 " " " " 77 "
3	60	2650	" 11 " " " " 55 "
4	60	2410	" 11 " " " " 55 "
5*	55	1930	" 10 " " " " 50 "
6*	55	1700	" 10 " " " " 50 "

* Nos. 5 and 6 made of blast furnace aggregate.

ABSORPTION TESTS OF CONCRETE USED IN PIPE.

Per cent absorption after 48 hours:
Stone concrete..... 4.4
Slag concrete..... 6.1

DISCUSSION.

Mr. Perrot. MR. E. G. PERROT.—I would like to ask Mr. Goldbeck how long the concrete sewers in Philadelphia to which he refers, have been in use and also the thickness of the egg-shaped pipe.

Mr. Goldbeck. MR. A. T. GOLDBECK.—I cannot say exactly how long the reinforced concrete sewers referred to have been in use; but certainly for ten years at least. The egg-shaped pipe was $3\frac{1}{2}$ in. thick, the circular pipe 4 in.

Mr. Johnson. MR. N. C. JOHNSON.—Did any corrosion of the reinforcement show on either the interior or exterior of the pipe? In a section so thin, when steel is in a position where it can corrode due to the various waters which flow through the sewer the concrete is in jeopardy, because of the expansive force of the various compounds which are formed during corrosion, the main one of which is common iron rust. The immense expansive force of corrosion is progressive, as every one knows.

Mr. Goldbeck. MR. GOLDBECK.—In the concrete pipe we have not had a chance to note any corrosion, as they have been made about 3 or 4 months only.

Mr. Bruner. MR. L. S. BRUNER.—The City of Halifax, Nova Scotia, has had concrete sewers in use 25 or 30 years, entirely for domestic sewage, and they have proven so satisfactory that the city has established a municipal pipe casting plant.

Mr. Wig. MR. R. J. WIG.—I would like to ask Mr. Goldbeck the condition of the pipes which were steam cured after the jackets were taken off. Were they dusted, wet or damp? Did they appear as though the water had been removed?

Mr. Goldbeck. MR. GOLDBECK.—I think they had a rather dry appearance.

Mr. Wig. MR. WIG.—Were they dry when broken?

Mr. Goldbeck. MR. GOLDBECK.—We didn't break them right after they were cured; the exterior was not bone dry, but had a dryer appearance than it should have had. They were about 12 hours old, and should have been wet.

Mr. Wig. MR. WIG.—Several years ago, we made an investigation of the effect of steam pressure on concrete pipe and we found if it

was mixed quite wet as in ordinary practice, and as probably mixed in the case of this pipe, we could not expose it to steam for the first 2 or 3 days or it would reduce the strength. But if it was mixed quite dry, as a concrete block made by the dry method, it could be exposed to steam within 12 to 24 hours and would show a very great increase in strength; but if exposed to steam immediately it would crack. It may have very well developed cracks or slight cracks which would not be noticeable. Mr. Wig.

MR. PERROT.—Regarding the protection of the steel by the concrete in sewer pipe, this might be apropos of that question. In the City Hall pavement in Philadelphia, which was laid some years ago, the top coat has $\frac{1}{4}$ in. square bars, and they are within 1 or $1\frac{1}{4}$ in. of the surface. One of those sections was removed some few years ago, and the man in charge of removing the sidewalk gave me a piece of rod taken out, which had been in 19 years. I have that rod in my office. It was $1\frac{1}{4}$ in. from the surface, and it showed no evidence of corrosion or rust of any kind. I understand from Mr. Goldbeck that the rods were within $1\frac{1}{4}$ in. of the side of the pipe. If the concrete is of the proper density I do not think there will be any trouble from rust. Mr. Perrot.

MR. D. A. ABRAMS.—I was going to call attention to the fact that the live steam in steaming concrete products, from the mere thermo-dynamic theory, would have a tendency to dry out materially instead of supply moisture, as might be expected. That is a thing which is overlooked quite often. I very recently had occasion to test a lot of concrete bricks which the manufacturer thought he was steaming; instead he was drying them out with steam which was not saturated. Mr. Abrams.

MR. GOLDBECK.—The cracks were discovered in a rather crude way, by the use of a magnifying glass, and I think it depended a great deal upon the vigilance of the observer. I remember in some cases where I did not see them, the other observers did, and vice versa. It is a question how large the cracks can be before the deterioration of the steel begins to take place. I certainly should not like to have a distinctly visible hair crack form in a concrete pipe. Mr. Goldbeck.

PROFESSOR A. N. TALBOT.—I have heard of a few cases where steel placed as close to the surface as this has suffered injury within a few years; not in the case of a sewer but in that of a vessel Mr. Talbot.

Mr. Talbot. exposed to water. Whether steel will corrode or not, seems to me to depend very much upon the quality of the concrete.

Mr. Collings. MR. W. A. COLLINGS.—Of course that could be obviated very easily by using galvanized metal.

Mr. Ferguson. MR. L. R. FERGUSON.—I would like to ask Mr. Goldbeck if the slag to which he refers was selected for this test or was it simply the ordinary slag bought in the open market?

Mr. Goldbeck. MR. GOLDBECK.—The slag used was a sample of material which ordinarily comes to the Philadelphia market. It weighed about 74.7 lbs. per cu. ft.

THE MANUFACTURE AND LAYING OF REINFORCED CONCRETE SEWER PIPE.

By Henry T. Shelley.*

During the year 1914 the Bureau of Surveys, City of Philadelphia, awarded contracts for the construction of reinforced concrete pipe sewers, combined system, designed to carry storm water run off and domestic sewage.

The plans of these sewers called for two sizes of pipe: 36 in. diameter circular and 36 in. by 24 in. egg shape; the alignments being long and straight with a number of regular curves of 30 ft. and 50 ft. radius for extensions in intersecting streets and for outlet connections.

The design of both sections called for one line of triangular steel mesh to be placed $1\frac{1}{4}$ in. from the inside face of the concrete shell; the shell for the 36 in. section to be 4 in. and for the 36 in. by 24 in. section $3\frac{1}{2}$ in. in thickness except at the base. The circular section was concentric throughout; the egg shape section was designed to have a base 8 in. in width.

Both sections were made in 4 ft. lengths with ends normal to the axis of the pipe; molded with socket and spigot ends, with the steel reinforcing projecting at each end a distance of $1\frac{1}{2}$ in., overlapped when laid to form an interlocking joint. Some 3 ft. lengths were made for the proper location of slants for house drainage connections and also for closures on curves. The curves were turned with 4 ft. and 3 ft. lengths with ends bevelled on a line with the radius of the curve, otherwise they were the same as for straight lines.

In this design the socket and spigot is formed in the shell of the pipe (without projection on the interior or exterior surfaces) so that when laid in the trench the projecting reinforcing can be overlapped in the socket and grouted. (Fig. 3.) The grouting being done from a pouring hole molded in the top of the socket end of each length. By this means a continuous and uniform

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thickness of section is maintained throughout the length of the sewer barrel.

The specifications required the concrete to be mixed in the proportion of 1 part cement, 2 parts sand and 4 parts crushed pebbles or broken stone. The steel for reinforcing was cold drawn, open hearth carbon steel having a tensile strength of between 80,000 lbs. and 90,000 lbs.; external loading and internal hydrostatic requirements were specified as was the character and

FIG 1.—MANUFACTURING AND STORAGE YARD. REINFORCED CONCRETE PIPE.

design of forms. It also was required that the pipe be made by an established manufacturer of reinforced concrete pipe.

A vacant lot, 60 ft. in width by 400 ft. in length, was secured convenient to the location of the sewers to be constructed, and a plant, Fig. 1, consisting of a one bag batch mixer operated by a 4½ H. P. gasoline motor, one 30 H. P. boiler, measuring barrows and forms provided. The lot had been excavated for cellars to a depth of about 6 ft. below street surface except a section about 100 ft. in length on the east end which was at a slight elevation above

street surface and which was large enough to accommodate the mixer and provide for the storage of materials. The lower section was utilized for the manufacture and storage of pipe. The location of the mixer provided descending grades for the wheelbarrow runs from the mixer to the forms. These runs were made in short sections and of a design that permitted their lengthening as the work progressed or removal to different locations as the several lines of forms required.

The forms consisted of cast iron bottom and top rings shaped to form the socket and spigot ends, with sheet steel inner and outer casings rolled to true cylinders. The inner and outer casings were concentric when assembled. The forms were held rigidly together with clamps and bolts to guard against cracks through the inner and outer rings, and to prevent the leakage of the mix. Butt straps were provided to cover joints between adjacent sheets.

The forms were assembled in four lines of 7 or 10 forms each with 6 ft. lanes between lines. The bases forming the socket end of the pipe were laid first to line and leveled carefully, a slot being molded in this base to receive the reinforcing. This slot after receiving the reinforcing was filled with sand to serve as a stop for the concrete and provide for the projection of the reinforcing metal in the socket.

The reinforcing was placed after the inner steel casings were assembled and clamped to the base; the assembling of the outer casings following immediately. All surfaces exposed to the concrete were coated with paraffin oil.

A light steel angle, bent to shape, and bolted to the exposed face of the inner steel casing, supported a thin steel plate covering the entire interior of the form. This plate or head served as a platform to receive the concrete from the mixer and from which it was evenly deposited and worked into the form.

The steel mesh was received in rolls of about 450 lbs. average weight and were of width sufficient to provide the reinforcing for the full length of pipe sections without splicing.

Portland cement, Jersey gravel or Jersey bank sand, and trap rock aggregate, graded in size from $\frac{3}{4}$ in. to $\frac{1}{4}$ in., were used in the concrete mixture. The proportions were by volume, although the mixture was occasionally changed to suit the grading of the sand and aggregate so that frequently a richer mixture

FIG. 2.—36 IN. CIRCULAR AND 24 X 36 IN. EGG-SHAPED SECTIONS, WITH 24 X 36 IN. MOUNTED ON CART.

was obtained than the 1–2–4 specified. Measuring barrows were used to serve the mixer which was a one bag mixer operated by a 4½ H. P. gasoline engine. Sufficient water was used in the mix to cause the concrete to flow readily when dumped on the steel head in the center of the form. As soon as mixed the concrete was turned into the receiving pan in front of the mixer, hoed into barrows, wheeled to the forms, and dumped on the steel heads, from which it was shoved into the forms uniformly and spaded.

This work of placing and spading was performed by one man using a spade made of a steel mortar hoe blade slightly curved, fastened to a steel rod 4 ft. long to which was fitted a wood handle 3 ft. in length. The man employed was skilled in the work, but with the exercise of every precaution, air pits on the surface of the pipe could not be entirely prevented. A whole line of forms were completed when once started. The forms were assembled in the morning and poured in the afternoon. The first 80 lengths of pipe made were allowed to harden in air; the casings being removed 24 hours after pouring, the pipe remaining on the base 3 days. Upon the removal of the casings it was frequently found that the concrete adhered to them. To overcome this adhesion each line of forms was subjected to a hot vapor bath for 12 hours. To provide this vapor, a pipe line leading from an upright 30 H. P. steam boiler was run over the center of each line of forms, a canvas covering entirely encasing the forms served to condense the steam. The casings were removed in 12 hours, but the pipes were allowed to remain on the bases for 3 days as in the air hardening process.

The uneven and great variation in temperature obtained at the top and bottom of the forms, unless the greatest care is exercised, makes this a very questionable process.

Tests were made to determine the age at which the pipe could be laid, and while excellent results were obtained at 7 days it was not considered advisable to lay it at that age and 14 days was made the minimum for laying.

A two horse truck (Fig. 2) was used to convey the circular pipe from the yard to the work; a one horse cart designed for the purpose being used for the egg shape pipes, each length was lined up along the trench in place and position for lowering when delivered.

One side of the trench was reserved for and kept clear to receive the pipe as delivered, the opposite side being used for the deposit of the excavated material. In this way it was possible to deliver a large number of pipes to the work before the trench was ready to receive them.

FIG. 3.—TWO SECTIONS OF 36 IN. CIRCULAR PIPE SHOWING SOCKET AND INTERIOR JOINT.

The plans required that the trenches be excavated to a width 12 in. greater than the horizontal external diameter of the pipe to be laid in them and to a minimum depth of 3 in. below the bottom of the pipe; this depth being increased when necessary to obtain a firm foundation. The design required all pipe to be supported in a concrete cradle covering the full width of trench

and extending from the bottom of the trench to a height of 12 in. above the bottom of the pipe. The concrete for the cradle was mixed in the proportion of 1 part cement, 3 parts Jersey gravel and 4 parts crushed stone.

Several methods for laying the pipe in this cradle were tried. The method adopted required the concrete to be placed in the trench to a depth sufficient to bring it to within $\frac{1}{2}$ in. of the ele-

FIG. 4.—HANDLING DEVICE USED FOR LAYING PIPE IN TRENCH.

vation of the bottom of the pipe when laid. This was allowed to harden for at least 3 days before pipe was placed on it. A bed of mortar about $\frac{5}{8}$ in. in thickness was spread over this base to seat the pipe and bring it to its proper elevation. After the pipe was laid and jointed the cradle was completed to the required height.

Immediately previous to lowering the pipe the mortar bed was spread for each length; the pipe being shoved over it and

FIG. 5.—24 x 37 IN. PIPE SEWER COMPLETED, READY FOR BACK FILL.

settled into place. Each length was laid with the socket end forward, to exact line and grade with a snug fit at the joint. (Fig. 5.)

The laying of the pipe was immediately followed by the "making up" of the joint. This was done by first overlapping the reinforcing projecting into the socket and pressing it into position, then applying a stiff mortar to the lower half of the socket from the inside of the pipe, and grouting the upper half from the outside through the "pouring hole" at the top. The mortar was pressed through the reinforcing until the lower half of the socket was completely filled when it was troweled to a smooth finish; a steel band 6 in. in width was then sprung around the entire interior of the pipe and keyed tight to prevent the escape of the grout, the exterior being sealed with a coat of stiff mortar covering the joint on the entire upper half of the pipe and extending about 12 in. below its center. While the grout was being poured a heavy steel wire was worked in the joint to prevent choking and to expel the air. This was continued until the joint was completely filled and the grout brought to the top of the pouring hole. The steel band was removed in 24 hours and the joint smoothed to a finish on a plane with the interior surface of the pipe. The mortar and the grout were made in the proportion of 1 part cement and 2 parts sand.

A very cheap and simple device was used to lower the pipe into place. It consisted of a light 6 in. steel I-beam 14 ft. long supported at the ends on two wood "A" frames resting on $2\frac{1}{2}$ in. plank runs. A small steel four-wheel traveler supported on the lower flange of the beam carried a differential pulley to which was hung a wood U-shaped carrier. The "A" frame consisted of two 6 by 6 in. yellow pine legs, with a 4 by 6 in. strut bolted 2 ft. above their lower ends to prevent spreading; the upper ends being fitted under the upper flange of the beam and bolted to the web. The traveler consisted of a pair of steel wheels fitted to each side of the beam close to the web and seated on the lower flange so as to move freely over it. A light steel frame suspended from these wheels carried a hook from which was suspended the differential pulley. (Fig. 4.)

The wood carrier was made of three pieces of 4 by 6 in. oak, 4 ft., 3 ft. 6 in., and 12 in. in length bolted together with $2\frac{3}{4}$ in. bolts. The short length serving to separate the long pieces when

bolted together and providing space between them for the entrance of the pipe shell. The pieces were bolted together on edge at one end; the ends being brought flush so as to make a variation in length at the free end. When in service the pipe was carried on the 48 in. piece and suspended by means of a heavy eye-bolt placed in the 36 in. piece, 10 in. from its free end. This piece extended over the top of the pipe a distance sufficient to bring the eye-bolt over the center of the pipe. To insure greater strength a band made of 1½ in. steel, ½ in. in thickness was fitted around the carrier at the inside end of the middle piece.

When placed in position for lowering the I-beam extended across the trench and over the pipe lined up beside it. The "A" frames were supported on 2½ in. plank runs extending along both sides of the trench. By means of the traveler the carrier was moved directly over the pipe, engaged to the pipe shell, which after being raised was moved by means of the traveler to a position directly over the center of the trench and lowered to place on the bed of mortar spread over the concrete base Before the carrier was disengaged each length was closely fitted at the joint and set to exact line and grade. The carrier was then raised, the "A" frames moved forward on the plank over the next length of pipe, and the laying continued. Pipe laying always commenced at the outlet end with the socket end forward. Where necessary to provide for manholes and spur connections an opening of 4 ft. or the length of 1 section of pipe was made. Where manholes were to be built the openings were excavated a width of 4 ft. 8 in. for a length of 6 ft., or 12 in. from each end of the pipe. The area of the concrete manhole base being of these dimensions.

A steel form shaped to fit the inside of the lower half of the concrete pipe and extending 12 in. in the pipe at both ends of the opening was used to form the invert of the manhole; the concrete being poured around the form to the center of the pipe, and after form extensions were placed, carried to an elevation of 7 in. above the inside of the pipe at the top. The inside of the manhole base at the top was oval in shape, the diameters being 3 and 4 ft. The 3 ft. diameter across the sewer being obtained for the egg shape section by narrowing the side walls to a thickness of 10 in. Above the concrete base the manholes

were built of brick 9 in. in thickness tapering to a 24 in. circle at the top or street surface. When connections for spurs were to be provided, wood invert and arch forms in addition to the steel invert form were used. These forms were assembled to form the entire connection before the concrete was poured, making a monolithic section throughout. This provided a square end on spurs for future pipe or brick extensions. The concrete mixture for manholes and spurs was the same as for the pipe.

Whenever possible, locations for pipe drains were determined and openings for their connection molded in the pipe. When the location of drains could not be determined before molding the pipe, openings for their connection were cut through the shell with diamond point chisels when the pipes were in place.

Several years previous, but when the cost of labor and materials were about the same, bids were asked and received for these sewers of the standard brick construction; the sections and plans being identical.

The lowest bid received for the 36 in. circular section of brick was $7.95 per ft. and for the 36 by 24 in. egg shape section of brick $4.35 per ft. The lowest bid received for the 36 in. concrete section was $5.10 per ft. and for the 36 by 24 in. egg shape concrete section was $4.72; a reduction of $2.85 per ft. for the 36 in. circular section and an increase of $.37 per ft. for the egg shape section. The price per ft. includes in both constructions excavation, manufacture, delivery, laying the pipe, back fill, removal of excavated material and repaving over the sewer trench.

In sewers of larger section a greater saving could be made and a better structure obtained by the use of reinforced concrete pipe than by any other method of construction; this is particularly true where water is present in the trenches in great volume and when its exclusion is imperative, as no other method presents equal possibilities of obtaining the same density and imperviousness of either the barrel or the joint at the same low cost.